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Vol. 1 Symposium Summary

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Held at

MASSACHUSETTS INSTITUTE OF TECHNOLOGY
Cambridge, Massachusetts

NOVEMBER 13-15, 1974

Sponsored by

Constructed Facilities Division
Civil Engineering Department
Massachusetts Institute of Technology

In cooperation with

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Department of Public Works

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16. Abstract

Reported are the Proceedings of the Foundation Deformation Predicition Symposium held at Massachusetts Institute of Technology, November 13-15, 1974. The purpose of the Symposium was to examine current design methodology for embankment loads placed on a soft soil foundation by comparing predicted and measured performance of a full scale field test.

Predictions of pore pressures, deformations and stability of the soft foundation under additional embankment load were made by ten teams of geotechnical engineers. These predictions were made before the conduct of the actual field test and are described. The results of the field test which included loading to failure of the foundation are given.

Volume 2, published as report FHWA-RD-75-516, is an Appendix to these Proceedings and contains the write-ups by the predictors.

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I. INTRODUCTION

A Foundation Deformation Prediction Symposium was held at MIT during November 13 - 15, 1974. The purpose of the symposium was to examine current design methodology through the comparison of predicted and measured performance of a soft soil foundation subjected to an additional embankment load.

Since 1967 the Department of Civil Engineering, Massachusetts Institute of Technology, has participated in research on the performance of a high embankment placed over a thick clay foundation. The Massachusetts Department of Public Works and the Federal Highway Administration have sponsored the research. The major objectives of the research have been:

1. To develop methods for predicting the performance of the clay foundation under an embankment load;
2. To measure the performance of the foundation during and after construction, and
3. To compare the measured and predicted performances.

A substantial amount of field performance information has been obtained and analyzed.

As a part of this research, additional fill was placed on the present embankment during late summer 1974 to cause large undrained deformations in the clay foundation (i.e., reaching limiting equilibrium) and measurements of deformations and pore pressures were taken. Ten engineers (or teams) submitted

predictions of key aspects of performance to two referees prior to the conduct of the field test.

All of the predictors were supplied the same basic information. Several predictors requested additional data. Whenever possible, these additional data were then supplied to all predictors. All predictions were given to an independent referee before the start of the field test. No predicted or measured values were revealed until the symposium.

The symposium brought together the group of predictors to focus their attention on a single problem rather than present an analysis of different problems. In this way, the maximum synergistic benefit could be obtained from the efforts of the group.

At the symposium, each predictor presented his approach to predicting performance at previously specified locations and his predictions. Measured values were then revealed. Each predictor commented on his predictions and approach in light of the field performance. Discussion of the predictions and measured performance by the audience followed.

These proceedings document the symposium including predictions, measurements and discussion. Written presentations submitted by the predictors are presented where available; otherwise, we have attempted to indicate the approach taken, the parameters used and the predicted values as recorded during the symposium. In the interest of preserving the flavor and character of the symposium, we have not tried to compile a polished set of proceedings.

Prof. T. W. Lambe opened the symposium by discussing the importance of predictions. Geotechnical work at MIT during the last two decades has led to formulation of the following important considerations:

1. Decisions are based on predictions.
2. Predictions are an important part of civil engineering.
3. Predictions must be evaluated to assist in the decision making process and to improve the prediction process.
4. Predictions are frequently inaccurate.
5. The major cause of inaccurate predictions is faulty and insufficient data.

Several key steps can be identified in the prediction process:

1. Determine the field situation (gather facts);
2. Simplify the field situation (organize facts);
3. Determine mechanism(s);
4. Select both Method(s) and Parameters;
5. Manipulate Method and Parameters--make prediction;
6. Portray prediction, and
7. Evaluate-Improve (during and after symposium).

To help classify predictions, we at MIT use the following descriptions:

Prediction type	When prediction made	Results at time prediction made
Type A	Before event	---
Type B	During event	Not known
Type C	After event	Not known

This symposium was organized to provide an opportunity to present, discuss and evaluate Type A predictions.

NOTE

The original instructions to each of the predictors did not call for the preparation and submission of a written report describing his prediction. This fact explains the variation in the detail and form of the contributions. For the sake of accuracy we chose not to redraft certain of the figures.

II. PROBLEM STATEMENT

Embankment Description

Figure II-1 shows a cross-section of the embankment constructed in 1967 to 1969 and the subsurface soil profile at Station 263. Plans were to widen a 300-foot long section of the embankment crest centered at Station 263 by dumping fill along the east slope until the slope was about 40°. Trucks were then to dump fill on top of the embankment in 1-foot lifts at an average rate of 1- to 2-foot height increase per day, excepting weekends. No compaction, other than that produced by normal construction traffic, was planned. The additional fill would come from a portion of the present embankment about 1800 feet away. Filling was to stop for a two-day period after 6 feet of additional fill had been placed on the embankment.

Predictions to be Made

The predictors were asked to predict the following quantities:

1. After 6 feet of fill have been added to 1969 embankment, predict:
   a. additional horizontal movement of SI-3 and SI-4 at elevations -30 and -70;
   b. additional settlement of SP-1;
   c. additional pore pressure at P-3, P-4, P-6 (maximum during two-day period after filling is stopped), and

d. heave at H-1, H-2.

2. Additional height of fill (T in Fig. II-1) necessary to cause a stability failure (differential deformation at crest $\geq$ 0.5 m constitutes failure).

3. Pore pressure at P-3, P-4, P-6 at failure. Figure II-1 shows instrument locations.

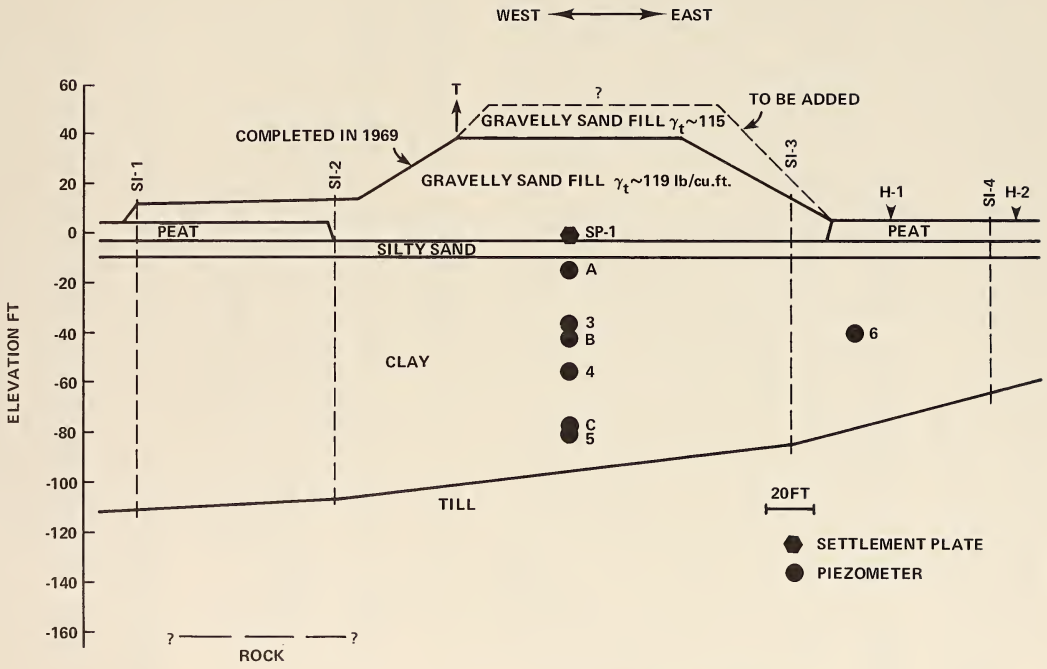


FIGURE 2-1. CROSS SECTION AT STATION 263

III. DATA SUPPLIED TO PREDICTORS

Information and data were sent to each predictor in a series of bulletins. This section summarizes the information given in these bulletins.

History of Initial Embankment Construction

Initial construction began in August, 1967 with removal of the top layer of silt and peat and placement of a 10-foot-thick sand and gravel working mat. During placement the fill was end-dumped with no compaction up to elevation +5 ft. Above elevation +5 ft, the fill was placed uniformly along the entire length of the embankment with scrapers and trucks. Rubber tired tractors and two 8,000 gallon water tanks (filled with salt water) helped compact the fill to an average total unit weight of 119 lb/ft^3.

The sequence of construction was:

August 1967 -- Peat removal and replacement commenced.

April 1968 -- Peat removal and replacement completed. Embankment filling commenced.

November 1968-- All filling stopped (Nov. 15).

April 1969 -- Filling resumed (April. 15).

May 1969 -- Filling 100% complete.

Geology of Area

The 5- to 45-foot-thick layer of glacial till found at the site was probably deposited during the Wisconsin glaciation over a gray shale bedrock found at depths of 70 to 204 feet.

Radiocarbon dates indicate that the Boston area became free of glacial ice at least 14,000 years ago and deposition of the clay was in progress at that time. Sediments of glacial origin were deposited in brackish water. Subsequently, during the Valder's glacial substage (12,250 to 11,750 years ago), sea level fell with respect to land. The clay sediments emerged from below sea level and were eroded by streams and weathered subaerially. The weathering and subsequent desiccation resulted in the formation of a stiff "crust" which exhibits the properties of overconsolidation. The Valder's glacial substage was followed by warmer climates during which sea level rose rapidly and sands were deposited in depressions in the clay surface. As the sea continued to rise at a slower rate to its present level, the entire area was covered by organic silt, shells and peat.

Preconstruction Soil Properties

The project lies in a low tidal marsh which is subjected to daily tidal inundations. The average ground elevation before construction was +5 feet (USGS). The average daily tide variation is 2.5 feet with average tide elevation at +2.5 ft (USCS).

Preconstruction soil properties of the clay from laboratory and field tests at nearby Station 246 are shown in Figure III-1. Very little preconstruction testing was done at Station 263.

The thickness of the natural sand layer overlying the clay varies with location. On the west side of Station 263, the sand grades into clay at elevation -12 feet. Borings at the centerline of Station 263 indicate the sand-clay interface is presently (1973) at elev. -18 to -20 feet while on the east-side sand ends at elevation -7 feet. Blow counts from standard penetration tests in the sand at Station 263 vary from 10 to 49. At Station 246 the sand blow counts are 8 to 25 with an average of 17. No laboratory tests were run on this material. Split spoon samples indicate that it is a well-graded, silty sand with some gravel.

No laboratory tests were run on the underlying till. It is quite dense with blow counts generally in excess of 50 but varying from 24 to 171. The following elevations have been determined for the top of the till near Station 263:

Location	Elevation (ft)
SI-1	-109
SI-2	-108
SI-3	-87
SI-4	-65
140 feet east of ℄	-68
At toe Sta. 263 +90	-80
At toe Sta. 263 +10	-87

The fill consists of well-graded fine to coarse sand, with some fine to medium gravel and a few boulders. The predominant mineral is angular quartz, but some mica is present. Figure III-2 shows the results of three drained, triaxial loading tests on the fill material.

1974 Soil Properties

A series of laboratory and field tests were conducted on the clay at Station 263 just prior to the addition of fill for the field test. Figures III-3 and III-4 show a summary of the results of these tests. All lab tests were run on samples trimmed from 5-inch diameter undisturbed samples obtained a few months prior to the start of the field test.

Field Measurements of Previous Performance

Three piezometers, four slope indicators and one settlement plate were installed during the early stages of construction. The three piezometers at the centerline (A, B, and C) ceased to function by 1971 and were replaced by piezometers P3, P4, and P5 in July, 1973, along with additional piezometers to monitor embankment performance during additional fill placement. The locations of these instruments are given in Figure II-1 and the measured results are sumarized in Figure III-5.

Publications Describing I-95 Research

The following is a list of professional papers which discuss aspects of I-95:

D'Appolonia, D.J., T.W. Lambe and H.G. Poulos (1971) "Evaluation of Pore Pressures Beneath an Embankment", JSMFD ASCE, Vol. 97, SM 6.

Lambe, T.W. (1973) "Predictions in Soil Engineering", 13th Rankine Lecture, Geotechnique, Vol. 23, No. 2.

Lambe, T.W., D.J. D'Appolonia, K. Karlsrud and R.C. Kirby (1972) "The Performance of the Foundation Under a High Embankment", Journal of the Boston Society of Civil Engineers, Vol. 59, No. 2, April.

Additionally, there are several MIT research reports documenting various aspects of the joint research effort and a series of bulletins giving more information on the soil properties at Sta. 263.

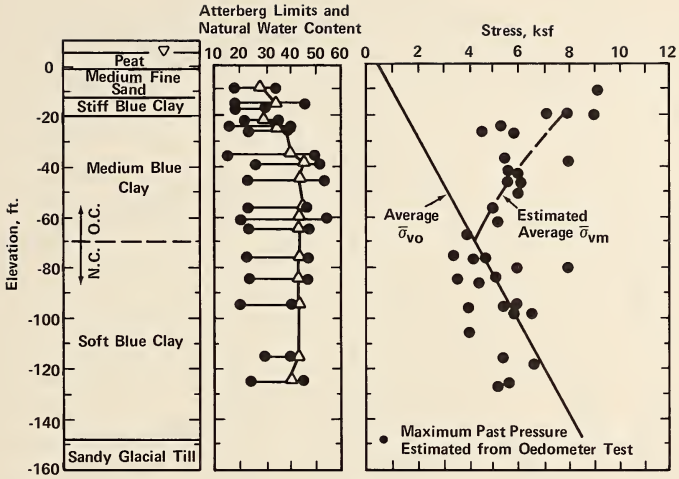


FIGURE 3-1. SOIL PROPERTIES OF MIT TEST SECTION, STA 246

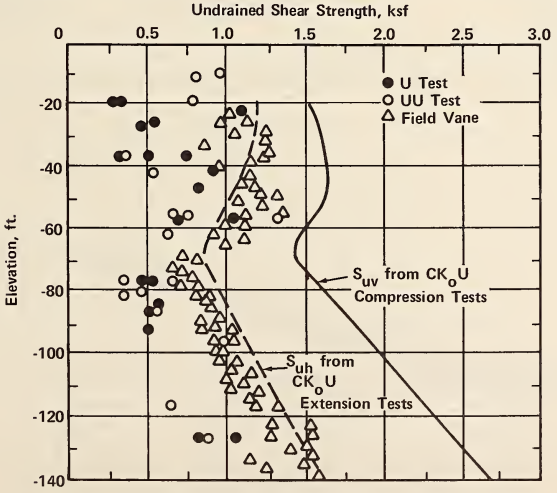


FIGURE 3-1. SUMMARY OF STRENGTH TESTS AT MIT TEST SECTION, STA 246
from D'Appolonia, Lambe, & Poulos (1971)

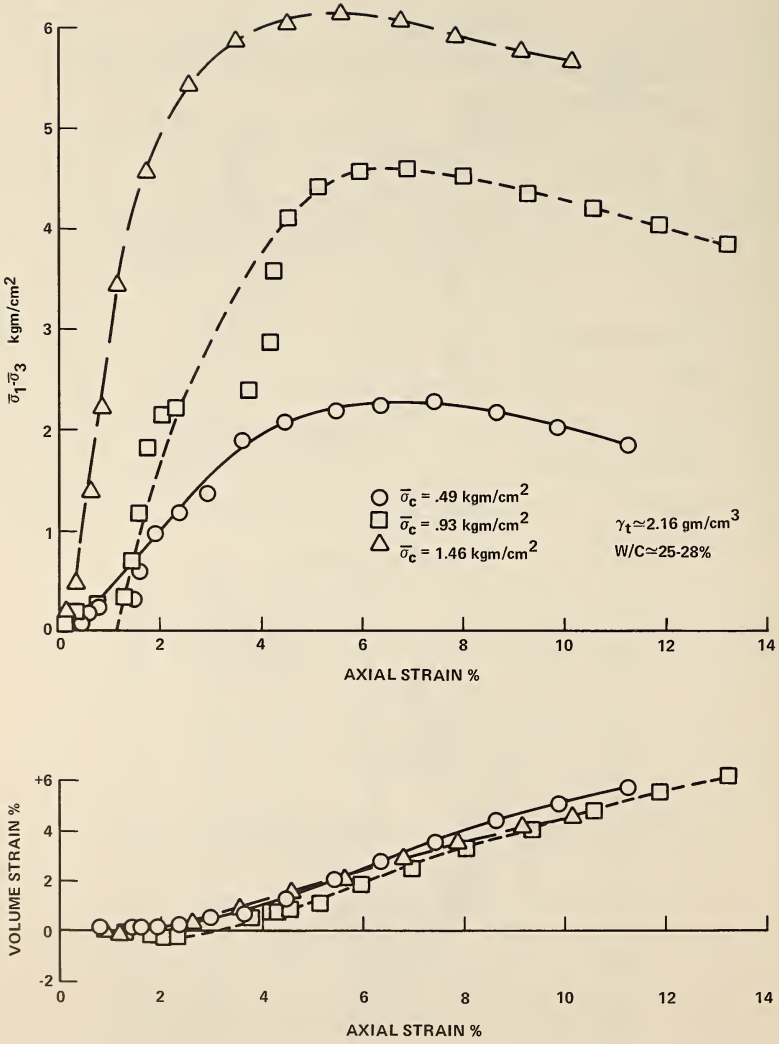


FIGURE 3-2. DRAINED TRIAXIAL COMPRESSION LOADING TESTS ON FILL

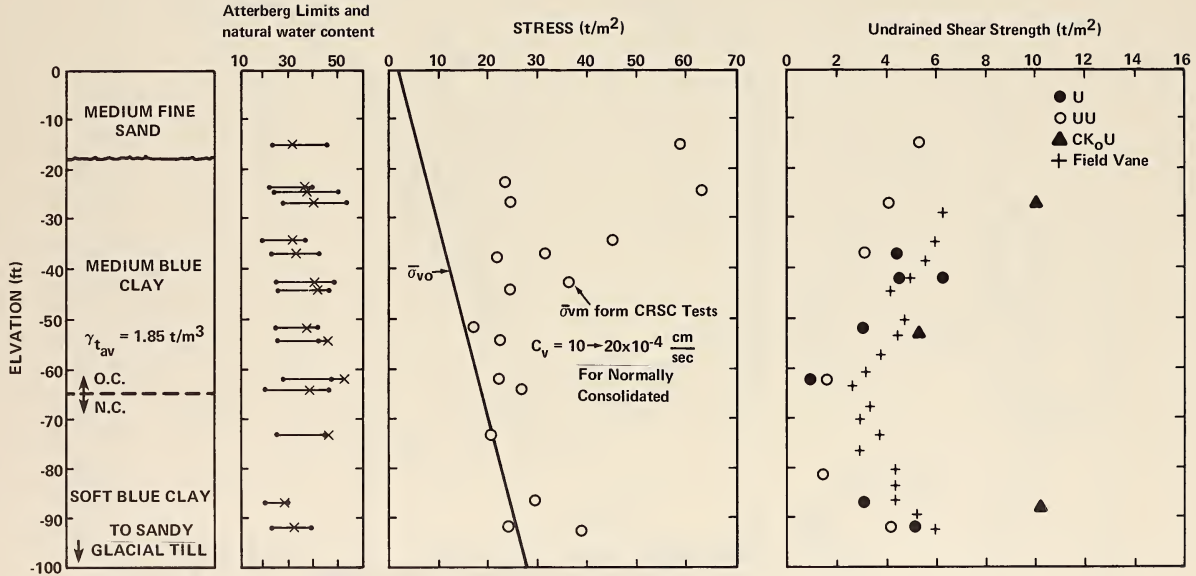


FIGURE 3-3a. 1974 SOIL PROPERTIES - STATION 263

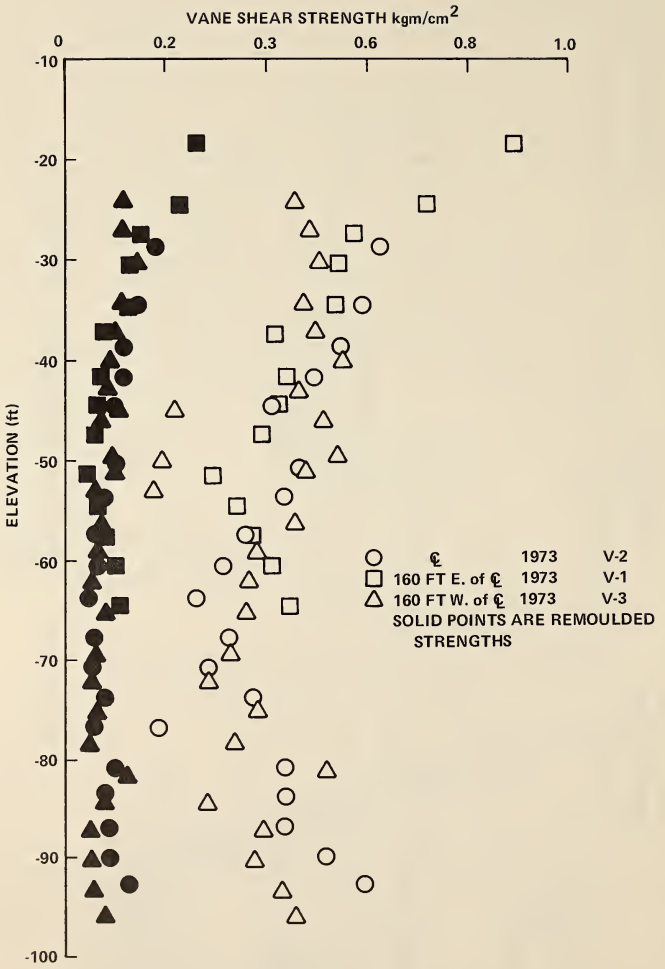


FIGURE 3-3b. STA 263+00 I-95

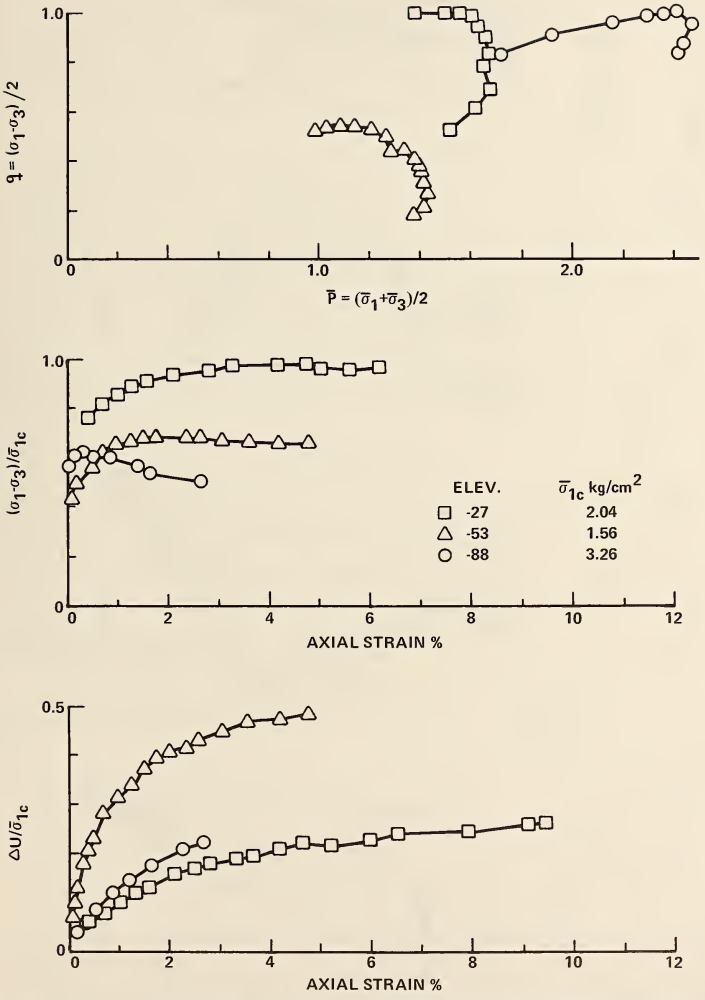


FIGURE 3-4. UNDRAINED TRIAXIAL COMPRESSION TESTS ON SAMPLES CONSOLIDATED TO PRESENT IN SITU STRESSES

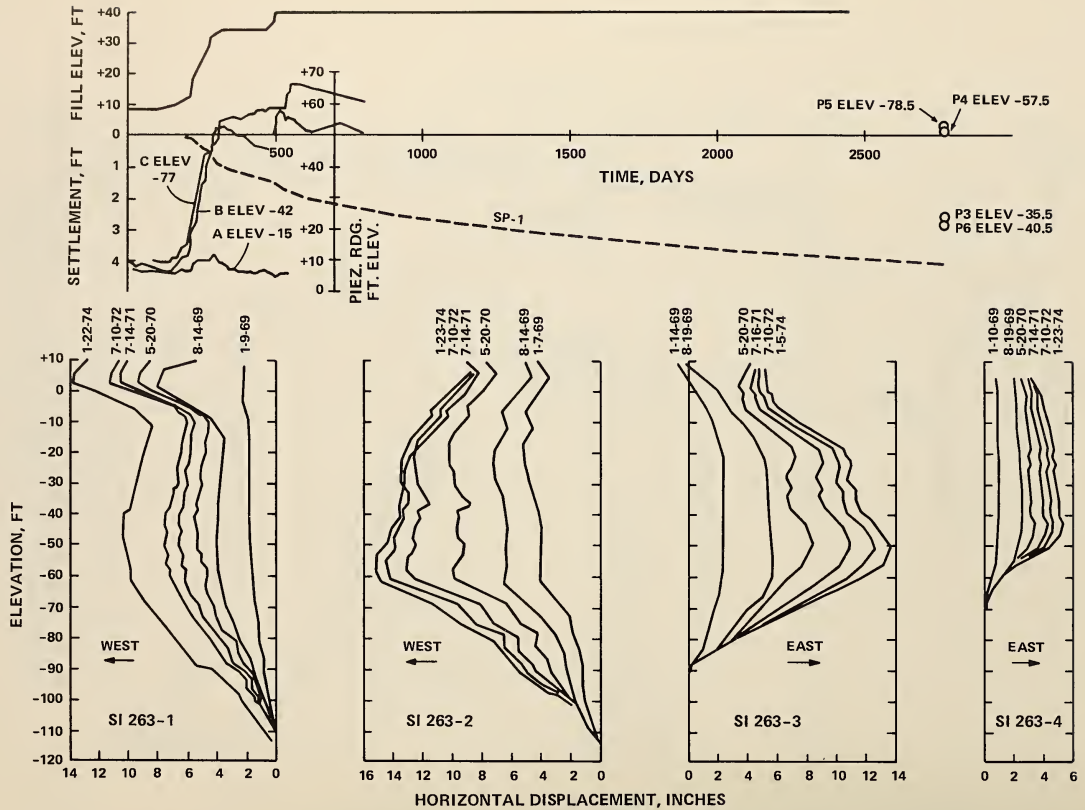


FIGURE 3-5. MEASURED PERFORMANCE STA. 263

IV. PRESENTATION OF PREDICTIONS

General

Field loading took place after the formal predictions were submitted, but prior to the symposium. Before disclosing the field results, each presented his predictions along with a discussion of the methods he employed. Several of the predictors used normalized soil properties (Ladd & Foott, 1974) and therefore, a discussion of this concept was presented by Prof. C. C. Ladd (one of the predictors) prior to the presentation of the predictions. A short biographical note for each predictor is given in Section VII.

Several predictors provided papers documenting their predictions. These papers are in Appendices A through H. The following pages present summaries of all the predictions.

H. G. Poulos and E. H. Davis (See Appendix A also)

Davis and Poulos based their prediction on two finite element computer analyses:

(1) An elasto-plastic analysis was employed to determine the undrained behavior of the embankment during construction. The finite element mesh for this analysis is shown in Fig. IV-1.

(2) An elastic consolidation analysis (Biot Theory) was used to estimate pore pressures and consolidation settlements both during and after construction.

In addition, several independent approximations and hand calculations were carried out to check the finite element calculations.

The prediction process was divided into two stages. Stage I predicted the performance of the embankment to its 1969 height for which field measurements were available. Stage II predicted the behavior of the embankment resulting from the recent addition of fill.

The soil parameters for the Stage I finite element analyses were originally adopted from laboratory and field test results and published data. Some of them were adjusted, however, by trial and error during Stage I so that the resulting predictions agreed reasonably well with actual observations. The range of values of all soil parameters are shown in Table IV-1.

In Stage II, allowance was made for the effects on consolidation which had occurred after 1969, but prior to the 1974 addition of fill, by determining, from the results of the finite element consolidation analysis, the changes in effective stress which had occurred since the start of embankment construction. The values of s_u and E_u just prior to the addition of fill were estimated on the basis of the current vertical effective stress, and hence the current OCR, and the correlations with OCR of the ratios $s_u / \bar{\sigma}_{vo}$ and $E_u / \bar{\sigma}_{vo}$ established in Stage I.

The contours of the estimated values of s_u and E_u before the 1974 addition of fill, as shown in Figures IV-2 and IV-3, were used in the finite element, elasto-plastic, undrained

analysis for the case of 6-feet additional fill. Predictions of deformations and additional pore pressures (calculated as the mean of the vertical and the horizontal stress increases) were obtained.

Further element layers were then added to the top of the embankment and general failure was assumed to occur when the rate of increase of plastic work became negative and convergence could not be obtained. The resulting predictions of the additional height of fill to cause stability failure and the other required predictions are listed in Table IV-2.

The UU test results were disregarded and the field vane test results were not directly used. Strength anisotropy of the Boston Blue Clay was considered significant and therefore, the average s_u was taken to be 90 percent of the average of s_u in the vertical and horizontal directions. The Stage I analysis also led the predictors to conclude that: (1) undrained creep could be a major problem in the Boston Blue Clay, (2) the till beneath the clay was effectively impermeable, and (3) hand calculation methods of approximation could give very good results, if accurate values of soil parameters were used.

W. W. Moore

Serving as head of the prediction team for Dames & Moore, Mr. Moore opened the presentation with some comments on the role of geotechnical engineering consultation services and his group's eagerness to participate in this event. Mr. M. J. Hess then took over, introduced the rest of the team--Mr. R. A. Bell and Mr. W. J. Roberds--and presented the analytical procedures

involved and the resulting Dames & Moore's predictions. The predictions developed by Dames & Moore used all of the subsurface information that was provided, as well as the various published articles on research work performed by MIT on the Interstate 95 embankment.

Strength properties of the Boston Blue Clay were assessed by the SHANSEP approach developed by Professor C. C. Ladd of MIT. Stresses induced by the embankment were computed by elastic theory. Pore pressure distribution was estimated assuming that change in pore pressure was equal to the change in octahedral normal stress, up to the point of local yield and at higher stresses the change in pore pressures was assumed equal to the major principal stress increment. The estimated values were compared with previous piezometer readings at the test section. The dissipation of pore pressure was assumed by using a coefficient of consolidation of 1.44 $feet^2$ per day and 0.14 ft^2 per day for overconsolidated and normally consolidated Boston Blue Clay, respectively for the stress ranges concerned in these analyses. The higher value for the overconsolidated clay applies only to stresses below the preconsolidation pressure. Drainage was assumed to occur at both the top and bottom of the layer. A profile of excess pore pressure existing as of August 1974 was developed from piezometer readings taken in the field at that time, as shown in the attached Figure IV-4. With these values of excess pore pressures, and the elastic stress analysis due to the embankment loading, a profile of vertical effective

stresses was developed, as shown in Figure IV-5. By using the effective stress distribution and the estimated maximum past vertical effective stresses (Figure IV-6), the shear strength profile as shown in Figure IV 7, was plotted by using SHANSEP theory. The strength of the fill was evaluated assuming $\phi = 40°$ in the existing embankment and $\phi = 35°$ in newly placed fill. The normal stress on the portion of failure surface in the fill was computed by elastic theory and also considered that an active state of stress existed. Stability analyses by both circular arc and sliding wedge were performed with the previously described strength parameters. Figure IV-7 shows the critical surfaces for both modes of failure. The circular arc analyses resulted in a factor of safety equal to 1.0 with the top of fill at elevation +55 ft. The results of the circular arc analysis were largely influenced by the strength of the fill. With the sliding wedge analysis, a factor of safety of 1.0 occurred with the top of fill at elevation +53 ft.

The creep behavior of the Boston Blue Clay was assessed in a qualitative manner when reviewing the results of the stability analysis. The assessment was based on past experiences of slope failures in San Francisco Bay mud. Even though the loading of the test embankment was to be in a relavitely short period, the effects of a possible delay were considered in an alternate prediction.

Failure of the embankment, if continuously loaded at a rate of 1.5 ft per day, was predicted at elevation +53. If loading

was delayed on the order of 5 to 7 days, failure was predicted at elevation +49 to +50. The latter prediction estimated the available shear strength of the Boston Blue Clay to be on the order of 80 percent of the ultimate. Laboratory tests to simulate the varying levels of field stress under variable duration of loading were considered very important to determining the height of failure.

Excess pore pressures were estimated by using elastic theory, with total applied stresses due to additional fill. The stress distribution charts by Giroud & Watissee were used with a Poissons ratio of 0.5. Solutions of applied total stresses were obtained at the points corresponding to piezometers P-3, P-4, and P-6 for the two loading cases of an additional 6 ft of fill and for an estimated 15 feet of additional fill. For the first loading, it was assumed that $\Delta u_e = \Delta\sigma_{oct}$, i.e., that no local yielding had yet occurred. This assumption was found to be consistent with the Mohr's circle representation of the stress state. The additional excess pore pressures for an additional 6 ft of fill were estimated to be 8.8, 7.2, and 4.8 feet of water for piezometers P-3, P-4, and P-6, respectively. Pore pressures for an additional 9 ft of fill (total of 15 feet) were assumed to vary with $\Delta\sigma$. At the predicted failure height, these values were estimated to be 16.6, 16.6 and 11.9 feet of water.

Deformations were predicted for an additional 6 feet of fill, which was kept on for two days. The additional vertical settlement at point SP-1 was estimated by elastic analysis, using the previously developed applied stresses and strength profiles. Assumptions were made that the Boston Blue Clay was linearly elastic within the loading range and that E_u = 1200 S_u. The additional settlement was estimated to be 1 to 1 1/2 inches.

The estimations of both horizontal displacement and heave were based primarily on the slope indicator data at Station 263 + 00 and a comparative study of the settlement, heave, and horizontal displacement measurements taken during loading of fuel storage tanks at a soft ground site in Mississippi and from a land reclamation project at the Redwood Shores Development in California. In the comparative studies, rough percentages of heave and horizontal displacement were compared to observed settlements during undrained loading. Horizontal displacements were thus empirically estimated to be 1/2 to 1 inch at elevation -30 and less than 1/2 inch at elevation - 30 were estimated for SI-4. Heave at 15 to 30 feet east of the toe of the slope was estimated to be less than 1 inch and at 65 to 70 feet east of the toe to be less than 1/2 an inch.

Table IV-3 presents the predictions.

C. C. Ladd

The work done by the MIT team was organized and directed by Professor C. C. Ladd. MIT had access to considerable background data for the I-95 site; however, their prediction was also submitted prior to the beginning of loading. Prof. Ladd's predictions are summarized in the following paragraphs. A complete report on the MIT prediction is presented in Appendix B.

Prof. Ladd's predictions were based on Normalized Soil Properties for Boston Blue Clay. The selection of strength and deformation parameters for the analyses is treated in detail in Appendix B. An accurate determination of the clay's maximum past pressure was particularly important for the selection of the proper normalized parameters. The finite element program FEECON was utilized for the deformation and pore pressure predictions. The initial construction of the embankment was first analyzed in order to "calibrate" the computer model with the measured field performance presented in the bulletins. This led to relatively minor changes in values of the moduli. A second FEECON analysis, which considered changes in properties due to partial consolidation under the embankment, was performed to obtain the required predictions. Figure IV-8 shows the input methodology for FEECON 2 and defines the four fill stages analyzed. Figure IV-9 shows the grid in the area of interest and the predicted zones of yielding beneath the left side of the embankment after adding 6 ft of fill (end of Stage III).

Figure IV-10 presents computed and measured deformation data for inclinometers SI-3 and 4. Line II is the measured

horizontal movement, Δ_H, during Stage II construction (elev. 34 ft to 40 ft). Measured increases in Δ_H over a 2.5-year period at constant fill height, presumedly due to creep, are denoted by Line C. From the information shown in Figure IV-10, it was concluded that creep deformations were very significant for the problem. Since there were no lateral deformations in the till, the best estimate of the predicted movements was taken as those computed by FEECON 2 at Elev. -30 ft and -37 ft minus those computed in the till. These values are shown at the bottom of Figure IV-10.

Plots of FEECON 2 computed heaves at the ground surface (top of peat at elev. +5 ft) and at the top of the clay (elev. -10 ft) vs. distance from the centerline are shown in Figure IV-11. Two elevations were considered since the location of the bottom of the heave stakes was not known. The best estimate was based on the average of the two curves.

Figure IV-12 presents a plot of fill elevation vs. centerline settlement at elev. -31 ft (original top of natural sand) and elev. -10 ft (original top of clay computed from FEECON). The predicted values, a combination of initial settlement due to undrained shear and consolidation settlement, are shown at the bottom of Figure IV-12.

Excess pore pressures were predicted using the equation:

$$\Delta u = \Delta\sigma_{oct} + a \Delta\tau_{oct}$$

Table IV-4 presents the predicted values of Δu at the three required piezometer locations after placing 6 feet of fill on the

existing embankment. A range of values of "a" were used based on the values back-calculated from field data for filling from elev. +35 ft to elev. +40). The pore pressures at failure were computed in the same manner.

Stability analyses performed to predict the added height of fill to cause failure concentrated on Morgenstern-Price (M-P) analyses using normalized values of undrained strength s_u. Undrained strength anisotropy was taken into account by varying the value of s_u according to the inclination of the failure plane as shown in Figure IV-13. Some circular arc analyses using corrected field vane strengths were made as a semi-empirical check on the M-D analyses. The results from the stability analyses are shown in Figure IV-14.

Professor Ladd's predictions are summarized in Table IV-5.

Z.C. Moh, E.W. Brand and A.M. Richardson

The team from the Asian Institute of Technology (AIT) was represented at the Symposium by Prof. Z. C. Moh. A complete report on the AIT prediction is included as Appendix C . Table IV-6 lists AIT's predictions.

The additional height of fill necessary to cause a stability failure was estimated using the Swedish slip circle method. Total stress analysis was used since it was assumed that the construction would be carried out essentially in undrained conditions.

A stability analysis was performed using an IBM 1130 digital computer. The computer programs were written to analyze the stability of the test embankment under the existing conditions taking into account the different subsoil strata and variations of strength values with depth. Variables introduced in the analysis were mode of failure, strength values of the foundation soils, unit weight and strength parameters of the embankment fill. The investigation of the stability of the existing embankment considered three modes of failure:

(1) Assume that the strength of the embankment and the underlying sand layer were fully mobilized along the circular slip surface;

(2) Assume that the strengths of the embankment and the underlying sand layer along the slip surface did not contribute to the stability, and

(3) Assume the strengths of the embankment and the underlying sand layer were fully mobilized as

being equal to the active earth pressure on a vertical plane multiplied by tangent of ϕ for the appropriate material.

The analysis for conditions at failure considered two modes of failure:

(1) Assume that the strengths of the fill, the embankment and the underlying sand layer were fully mobilized as being equal to the active earth pressure on a vertical plane multiplied by tangent ϕ for the appropriate material and

(2) Assume that the strengths of the fill, the embankment and the underlying sand were not mobilized.

Based on data provided, the following strength parameters were adopted:

(1) granular embankment fill: $\phi = 42°$ from drained test data.

(2) sand layer beneath the embankment: $\phi = 38°$.

Since the additional fill will be placed in a loose condition, the effective angle of shearing resistance was taken as $\phi = 36°$. We assume that the sand fill will have a placement water content of 10 to 12% and a total unit weight of approximately 106 lbs per cu ft. Bulletin 3 gave the total unit weight of the existing embankment as varying from 102.1 to 134 lbs per cu ft with an average of 119 pcf.

The additional fill would be placed in a short period of time. Thus, the foundation stability analysis was considered

an undrained case. The total stress analysis method was adopted. The strength parameters chosen were from field vane tests and from $\overline{CK_oU}$ compression and extension tests. No correction was made to the field vane values and strength anisotropy was considered by using the average values of shear strength in the horizontal and vertical direction. The required movements were predicted assuming that the deformations resulting from the additional fill would occur at essentially constant volume. Two different methods of analyses were considered:

(1) Analysis based on the theory of elasticity (Boussinesq solution) and

(2) Analysis using the finite element technique.

Neither method is considered an exact solution. In the elastic analysis, a value (or values) must be assigned to the modulus of elasticity (E) and Poisson's ratio (u). Poisson's ratio was taken as 0.5 for saturated clay. In the finite element approach, E, u and yield stress were selected for an elastic plastic stress versus strain relationship. Furnished data which were used to arrive at pore pressure parameters consisted of the piezometer B and C readings and the $\overline{CK_oU}$ triaxial compression test results.

METHODS EMPLOYED

Additional Height of Fill

The strength of the clay layer used in the stability analysis was developed from data from field vane tests at Sta. 244 and the section to be loaded (Sta. 263) and from laboratory CK_oU and CIU tests run on samples from Sta. 263 and Sta. 246. As can be seen in Figure IV - 15, vane strengths at Sta. 244 measured in 1972 show essentially the same profile that existed in 1967 prior to construction. It was assumed that the vane strengths at Sta. 263 had also remained essentially constant with time. From CIU tests run with a consolidation pressure four times the maximum past pressure (P_p) of the sample a shear strength ratio $C/\overline{P}_o$, for the normally consolidated clay was determined to be 0.28. It was assumed that this ratio was the same for CK_oU conditions based on experience with New York State clays where a maximum difference of 5% has been noted between strengths measured by the two test procedures.

From consolidation test data, the profile of maximum past pressure marked P_p (1967) in the left half of Figure IV - 16 was developed by discounting low values by assuming they resulted from sample disturbance. Since the clay below elevation -60 ft is thus shown to be slightly overconsolidated, a shear strength ratio of 0.33 was used to develop the strength curve marked CK_oU in the right half of Figure IV - 16. This value is the same factor obtained at the centerline of Sta. 246 in the slightly overconsolidated range.

The appropriate strength data for the overconsolidated region above elevation -60 feet was developed using the precompression ratio ($P_p/\bar{P}_o$) with Figure IV-17 to determine the strength ratio (S_u/C_u) of the normally consolidated strength to the overconsolidated strength at the same effective stress. This strength ratio (S_u/C_u) is then multiplied by the shear strength ratio ($C/\bar{P}_o$) and the existing effective stress to determine the strength. Figure IV-17 shows a band of values which has been developed with experience for New York State clays. The precompression ratio-strength ratio data from Sta. 246 is plotted within the band shown.

It was concluded that the vane strengths could remain essentially constant due to a cancellation of strength gains due to consolidation and strength losses due to long term undrained strains and that since the clay is in a state of plastic flow, the CK_oU strength would be too high. Stability analyses were performed using three different assumptions for the strength of the clay layer:

1. Use the vane strengths (Figure IV-16) with no allowance for strength gain.
2. Use the average of the vane and CK_oU strengths as shown in Figure IV-16 with no allowance for strength gain.
3. Use the average of the vane and CK_oU strengths adjusted to reflect the strength gain due to consolidation.

The analyses were performed using a Modified Swedish Wedge Analysis. Figure IV-18 shows the critical wedge for Case 1 noted above. A circular arch analysis was also run using Bishop's Method which yielded approximately the same results.

Movements Due to Adding Six Feet of Fill

The predictions of horizontal movement of slope indicators SI-3 and SI-4, the settlement of SP-1 and the heave of heave points H-1 and H-2 were all derived through interpretation of the data from the filling of the last six feet of fill placed in 1969. The movements of the slope indicators at the required points and the settlement plate due to the previous filling are shown in Table IV-7. Since the change in load to the foundation due to the additional loading would be less than the previous loading due to lower unit weight of the new fill material and a narrower crest, the measured deformations were divided by a factor of 1.18 (39/33) to account for the lower load. Similarly, a correction factor (0.9) was applied to account for larger deformations due to the increased rate of loading anticipated. The horizontal deformations and settlement arrived at from applying this procedure are also shown in Table IV-7.

Since no heave measurements were made during the previous loading, predictions of heave were made using the horizontal deformations of slope indicators SI-3 and SI-4 near the surface. Because the Bishop's circle gave a consistent result for stability, it was used to extrapolate the horizontal deformations to estimate the heave as shown in Figure IV-19. From the center of the critical circle near surface slope indicator points were

found so that the rotation at SI-3 and SI-4 would be the same to produce the measured horizontal movement. The vertical movement (heave) associated with that rotation was then plotted together with the assumption of no heave below the critical center and no movement in the canal to derive the heave vs. distance plot shown at the bottom of Figure IV-19. The predicted heaves for points H-1 and H-2 were then determined from the derived plot.

Additional Pore Pressures

The pore pressures were predicted based on analysis of the pore pressure response beneath the center-line during the original loading. Figure IV-20 shows the piezometer data extrapolating back to the anticipated pore pressure under instantaneous loading. It was felt that the data from Piezometer A were probably in error on the low side and since Piezometer C was damaged, the data from Piezometer B were used in analyses. Table IV-8 shows the change in Piezometer B which would have occurred due to instantaneous loading for each of the three previous loading periods and the increase in load with two assumed values of fill density. From this procedure, it was then concluded that the pore pressure response as measured in the piezometers would be approximately equal to the increase in total stress for an assumed instantaneous loading. Since the loading would be much faster than in the previous loadings, pore pressure predictions for both the additional 6 feet of fill and at failure were made on the assumption that the increase in pore pressure would equal the increase in effective stress.

PREDICTED QUANTITIES

Additional Height of Fill

From the stability analysis outlined above, a plot of additional fill height versus safety factor was determined for each of the assumptions with regard to shear strength as shown in Figure IV-21. After consideration of the failure mechanism and the vane strength data at Sta. 244, it was decided that the field vane data would give the best prediction. Accordingly, the height of fill to cause failure was predicted as 13.5 feet.

Movements Due to Adding 6 Feet of Fill

The calculated movements of the slope indicator points shown in Table IV-7 were based on the assumption that there would be no structural improvement of the foundation soils associated with a strength gain. However, in estimating the deformations, it was felt that some improvement would take place at least for Slope Indicator, SI-3. Accordingly, the calculated values were modified as follows to give the predicted deformations:

	Calculated	Prediction
SI-3 (-30')	1"	0.25"
SI-3 (-70')	1"	0.25"
SI-4 (-30')	0.25"	0.25"
SI-4 (-70')	0	0

Similarly, the calculated settlement of 3.5" was decreased to a predicted settlement of 3" due to some structural improvement. The predicted heave quantities at points H-1 and H-2 were rounded to 1.0".

Additional Pore Pressures

As noted above, the pore pressures were assumed to be equal to the change in total stress:

u (ft)	at 6' of fill	at failure (13.5')
P-3	7.3	14.8
P-4	7.2	13.9
P-5	4.9	7.6

DISCUSSION

The presentation was divided into two parts with Mr. McGuffey presenting generalized information on the embankment design process and the practice of the Bureau of Soil Mechanics of the New York Department of Transportation and Mr. Gemme presenting the prediction methods and results. In Mr. McGuffey's presentation, he stressed the importance of experience and the prediction was influenced substantially by the predictors' experience with Albany Clay which was reported to be similar to Boston Blue Clay. In discussing the design process, he noted that analysis is only one of several factors contributing to the final predictions and design and by no means the largest portion of the effort nor the largest source of error. Among the other factors in the design process, he mentioned the recent history of the site and noted the possibility that the foundation was more compressible than the section would indicate due to incomplete removal of the peat during foundation preparation. However, no apparent consideration was given to this point in preparing the centerline settlement prediction.

The consideration of strength gain due to consolidation of the embankment was a critical factor in the prediction made (see

Figure IV-21). It was not considered in determining the height of fill and consequently the pore pressures at failure nor for the estimate of heave. It was, however, considered in the case of lateral movements and to a lesser extent in the settlement of SP-1. From the measured results, it is apparent that a consistent application of a gain in strength and structural improvement due to consolidation effects would have improved (strength gain) or weakened (no strength gain) the prediction. The McGuffey-Gemme predictions are summarized in Table IV-9.

J. M. Duncan

Prof. Duncan based his deformation predictions on finite element computations, his stability prediction on limiting equilibrium methods and his pore pressure predictions on evaluation of the field data. Professor Duncan's predictions are summarized in Table IV-10.

Consolidation of the Clay

Initially, computations were made to determine the percentage of primary consolidation prior to new construction and using the results of these calculations, the effects of undrained strength and deformation properties of the clay were estimated. Settlement calculations showed that at most (assuming full consolidation) only 1.6 ft of the observed 3.8 ft of settlement could be accounted for by consolidation. The remainder was undrained movement, much of which was time-dependent. Creep effects, therefore, were thought to be very important for Boston Blue Clay. Because of the large amount of settlement due to undrained creep, the measured pore pressures were used to estimate the degree of consolidation. By varying c_v (as a function of OCR), the elevation of the static water table, and the initial excess pore pressure, good agreement between computed pore pressures and observed pore pressures (for July, 1974) was obtained. The conclusions from this matching were:

1. $c_v = 35\ (OCR)^{1.5}\ ft^2/yr$
2. $\Delta u_{excess} = 1.25\ \Delta\sigma_1$

3. The effective, average, static, water table elevation is +2.5 ft MSL.
4. Of the 3.8 ft observed settlement, 0.5 ft was due to primary consolidation.

Creep

Since no published data for creep tests on Boston Blue Clay were available, Professor Duncan requested undisturbed samples so that he could conduct his own creep tests. Two $\overline{CIU}$ tests were performed: (a) the first was loaded to a deviator stress corresponding to 40% of the short-term strength, and (b) the second was loaded to a deviator stress corresponding to 60% of the short-term strength. These loads were maintained for three weeks after which the observed strains were 2.5 to 3 times the measured strains at the same stress levels in a quick test. To account for these larger strains, Professor Duncan reduced the value of the initial modulus, E_i, used in the analyses. For a quick $\overline{CIU}$ test, the observed value of E_i/s_u was approximately 800. The one-day creep test gave $E_i/s_u \simeq 400$ while three weeks of creep resulted in $E_i/s_u \simeq 250$. For the finite element analyses, Professor Duncan used $E_i/s_u = 400$. The value of s_u used in the calculations was an average based on relationships given in Bulletin 3 (after Davis and Christian, 1971).

Finite Element Model

A symmetrical mesh, Figure IV-22, was constructed for preliminary work. The original intention was to use a complete non-symmetrical mesh later. Close agreement between observed

and calculated results (Table IV-8) led to use of a simple correction procedure and the symmetrical mesh instead of a larger, unsymmetrical mesh. In the finite element analyses, the hyperbolic stress-strain model was used for both the clay and the sand. The clay was characterized by values of E_i/s_u= 400 and $s_u/p = 0.7\ s_{uv}/p$. For the sand, the test data provided by MIT were inconsistent and estimated values based on data for similar sand were used in the calculations. The same properties were used for the natural sand, the sand working mat and the embankment sand. Very low stiffness was assigned to the peat.

The predicted values of settlement and horizontal movement due to 6 ft of additional fill were calculated using values of clay strength corrected for the increased effective stress resulting from consolidation under the weight of the embankment. Deformations were computed using the finite element method and correcting the computed values with factors obtained from earlier comparisons. Values for additional pore pressures were computed using the relationship $\Delta u = 1.25\ \Delta\sigma_1$ with the $\Delta\sigma_1$ taken from the finite element analyses.

Stability

Professor Duncan expressed the opinion that the finite element method, with hyperbolic, elastic stress-strain relationships cannot be expected to give reliable results for conditions near failure. The hyperbolic stress-strain relationship is useful for calculating movements at moderate stress levels but is not a good method of predicting stability and failure loads. The

stability computations for his prediction of failure height were made using the Modified Bishop Method of analysis with undrained strengths for the clay and drained strengths for the sand.

To determine how creep affected the ultimate strength of the clay, both the 40% and 60% $\overline{CIU}$ tests were loaded to 90% of short-term strength and both failed within 1 1/2 days. The strengths used in the stability calculations were the same averaged s_u values used in the finite element calculations but reduced by 10% to account for loss of strength due to creep. These reduced strengths were not used in the determination of E_i for the finite element analyses since the creep test strengths were not available when the finite element analyses were performed. The calculated values of factor of safety were plotted against fill height for both full s_u and 90% s_u accounting for 10% creep strength loss (Figure IV-23). The predicted value of embankment height at failure was chosen from the 90% s_u curve at a factor of safety equal to 1.0, and then discounted from 14 ft to 12 ft of additional fill because Professor Duncan suspected that the laboratory tests might result in strengths which were higher than the field strengths.

Failure Pore Pressures

After prefacing his remarks with a disclaimer on the usefulness of pore pesssures in an undrained analysis, Professor Duncan presented his failure pore pressures. They were computed using the $\Delta u = 1.25\ \Delta\sigma_1$ relationship obtained from the consolidation computations. The change in maximum principal stress was computed using an additional fill height of 12 ft.

Summary

Professor Duncan's major concerns were how to account properly for creep effects in Boston Blue Clay. Using the results of the two creep tests he had performed as a guide, he reduced the slope of the stress-strain curve and the value of s_u. His predictions were based on the finite element method (modified on the basis of comparisons between measured and calculated movements), an evaluation of field data (checked with consolidation computations for pore pressures) and limiting equilibrium analyses of stability.

Appendix D presents a more detailed description of Professor Duncan's predictions.

R. C. Kirby

The approach used by this team, Woodward-Moorhouse and Associates, Inc. (WMAI) was that of a consulting engineer, considering the researchers to be the client. The clients' "needs" were predictions of deformations, pore pressures and the failure load on the clay. A detailed treatment of WMAI's prediction is found in Appendix E.

It is likely that a consultant would make the prediction of deformation and pore pressures due to the addition of six feet of fill by first replotting the available field measurements data and then extrapolating them (using judgment). The prediction of the height of fill to cause failure would be considered as a practical tool only if extrapolating the measured data was considered very uncertain, and the consequences of being wrong included a severe cost penalty or an unacceptable risk to the client.

In order to arrive at the needed predictions and also evaluate methods of prediction performances, the project approach shown on Fig. IV-24 was adopted. The key characteristics of this approach is an interim evaluation of the Finite Element Method by determining if it could successfully hindcast the measured performance of the embankment previously constructed to elev. 40. This project approach was developed by the project engineer (Mr. Kirby) and reviewed and approved by senior level members of WMAI. The judgments required at the decision points were made by the project engineer; these judgments were also reviewed.

If the Finite Element Method proved satisfactory for hindcasting performance, it would be used as a prediction tool in combination with other methods. If the Finite Element Method proved unsatisfactory, predictions would be made by extrapolating the measured data and the use of limiting equilibrium methods. Although not shown on the project approach diagram, "quick and dirty" checks of the analyses were included whenever possible.

WMAI was unable to hindcast the measured performance of the embankment constructed to elev. 40 using the finite element method. The details of the finite element analysis are included in Appendix E. The predictions are summarized in Table IV-11.

Movements and pore pressures were predicted by first interpreting and then extrapolating the measured settlement, horizontal movement and pore pressure.

Construction from elev. 10 to elev. 34 required approximately five months to complete and a portion of the settlement measured during construction is due to consolidation of the clay. On the other hand, the addition of six feet of fill in September 1974 proceeded at a rate approximately five times faster than the construction from elev. +10 to elev. +34. Consequently, there was a need to separate the undrained or immediate settlement from the total settlement. A rate of settlement analysis was made using the computer program PROGRS (Schiffman and Stein, 1969) in order to determine what the shape of a theoretical settlement vs. square root of elapsed time plot would be. The rate analysis included the effects of loading history, variation of c_v with depth, and approximate corrections for two-dimensional strain and drainage.

Both piezometers (where predictions were required) were approximately 20 feet from the nearest drainage layer and dissipation of pore pressure during construction at these locations was considered negligible. The measured horizontal movements during construction were probably affected somewhat by consolidation of the clay during construction. However, no reasonably simple means of investigating this effect are available and no corrections were made.

Recognizing the uncertainty of any prediction, it was decided to estimate the deformation caused by adding six feet of fill in 1974 to be the same as the measured deformations due to raising the embankment from elev. 34 to elev. 40 in 1968. The predicted additional pore pressure at piezometers P-3 and P-4 are based on extrapolating the measured pore pressures at piezometers A and B. These data show a two-feet increase in total head for each one-foot increase in fill elevation. For the additional 6 ft of fill, a 12-foot increase in total head at both piezometers P-3 and P-4 was predicted. The additional pore pressure at P-6 (which is located near the toe of the embankment) could not be predicted by extrapolating measured data during construction since this data does not exist. The additional pore pressure was estimated analytically from the change in average total normal stress at that location which was determined from the finite element analysis. The same logic described above for extrapolating the settlement data was applied to extrapolating the horizontal movement data. No prediction was made for the

heave at H-1 and H-2. This prediction can only be made analytically because no previous measurements are available. WMAI's analytical model (the finite element method) proved unreliable for predicting deformation, and other simpler analytical methods were deemed inappropriate.

The plane strain active and passive strengths are considered a best estimate for the preconstruction undrained strength but have not been verified by full-scale local tests to failure. Analyses were made using the ICES program LEASE (Bailey and Christian, 1969). The program considers circular failure arcs and computes the factor of safety using both the Fellenius and Simplified Bishop method of slices.

Effective stress parameters were input for the embankment fill and natural sand ($\bar{c}$ = 0, $\bar{\phi}$ = 40°) and undrained shear strengths were used for the clay. The analyses were made for three different profiles of undrained shear strength: Case 1--field vane strengths; Case 2--preconstruction plane strain active and passive strengths, and Case 3--plane strain active and passive strength increased to approximately account for the increase in undrained shear strength due to consolidation.

By analyzing the performance of the embankment constructed in 1968 and 1969 to elev. 40 using the finite element method, WMAI could not hindcast the measured behavior without making major revisions in the input parameters. The input parameters chosen for the finite element analyses are considred best estimates and major changes in these parameters are not justified.

Moreover, because of the many parameters which could be changed to give good agreement between measured and predicted performance, the significance of such agreement would be questionable.

G. P. Raymond

The prediction of performance of the embankment founded on Boston Blue Clay made by Dr. Raymond was essentially based on a numerical model: a finite element program. The strength properties used for the undrained analysis were obtained from field vane shear strengths with no reduction to account for PI. The results of the vane tests, together with the results of unconfined compression and UU tests are presented in Figures IV-25 and IV-26. Fig. IV-26, in addition, contrasts the field vane strength prior to any construction to the strengths after Stage 1 was built. This comparison seems to indicate no increase in shear strength with time after the construction of Stage 1. The straight line on both figures represents the assumed soil strength. Past experience with Canadian clays has suggested that the results of laboratory tests performed on undisturbed samples consolidated to pressures higher than the maximum preconsolidation pressures are not indicative of the soil in situ properties. Consolidating to higher pressures than $\bar{\sigma}_{vm}$ breaks the soil structures and changes the soil properties. Strength parameters from field vane tests on Canadian clays are more representative of real in situ properties than results based on laboratory tests that follow the technique described above.

The stress-strain properties needed for the lateral movement predictions performed by the finite element program were obtained from the New Liskeard $\left(\frac{\sigma_1-\sigma_3}{\sigma_1-\sigma_3}_f\right)$ vs. strain data and the MIT

vane results. Because of time considerations, some of the laboratory tests requested could not be performed. The tests requested were undrained test, consolidated isotropically to 1000 psf, sheared to 4000 psf, expanded to 3/4 of the effective overburden and finally sheared well beyond failure (to about 20% strain). Data from these tests was also needed for pore pressure predictions.

The New Liskeard failure index vs. strain is presented in Fig. IV-27. Boston Blue Clay was suspected to have higher strength than the New Liskeard varved clay. For this reason, the New Liskeard strains were divided by a suitable factor to fit the MIT data. This was accomplished with a factor of $E_{(MIT)} = E_{(NL)} / (1.5)$.

The strength parameters of the fill used were $\phi = 37^{o}$ and $\gamma = 120$ pcf. A similar analysis on an embarkment built near Kars, Ontario, suggested that the embarkment stiffness should be neglected in the stability analysis. This approach was followed.

According to the finite element analysis mode for 6 ft of additional fill, all the piezometers had either failed or were very close to failure. To predict the increase in pore pressure, a value of A equal to 1.0 together with Skempton's pore pressure equations were used for all the piezometers. In addition to this, for those piezometers that had already yielded, a range of predicted pore pressure was obtained by computing an upper value based on an empirical equation. The equation used is as follows:

$$\Delta U = \bar{\sigma}_{vo} + \Delta\sigma_v - \frac{3Su}{\gamma w} - U_o$$

where $\bar{\sigma}$ = effective vertical stress prior to the addition of 6' of fill

$\Delta\sigma_v$ = increment in total vertical load

S_u = undrained strength as computed using field vane.

w = unit weight of water

U_o = total pore pressure prior to the increment in load.

The same approach was used for predicting pore pressure at failure.

Conclusions:

I. Additional fill to cause failure:

a. Model used

1. numerical model based on finite element approach

b. Strength Parameter

1. Obtained from field vane test with no correction to account for PI

c. Embankment stiffness neglected

d. Prediction: 8 feet of additional fill.

II. Additional Movement for 6 feet of fill

A. Model used

1. finite element program

B. stress-strain data

1. Obtained from New Liskeard $\frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f}$ vs. E together with the vane results.

2. Correction on NL strains was made to account BBC higher strength.

C. Prediction of lateral movements:

1. SI - 3 at - 30 0.72 ft.

2. SI - 3 at - 70 0.72 ft.

3. SI - 4 at - 30 0.44 ft.

4. SI - 4 at - 70 0.00 ft.

D. Predictions of movements of surface

1. At center line 1.14 ft.

2. At heave pt. H-1 0.40 ft.

3. At heave pt. h-2 0.52 ft.

III. Additional Pore Pressure for 6 ft of fill

A. Model

1. Skempton pore pressure equation--lower value

2. $\Delta V = \bar{\sigma}_{vo} + \Delta\sigma_v - \frac{3s_u}{\gamma_w} - U_o$ --upper value

B. Parameter

1. A = 1.0 (assumed)

2. B = 1.0 (saturated soil)

3. s_u = field vane test

4. Stresses and existing pore pressure as predicted by finite element program.

C. Prediction for 6 ft of fill:

1. P-3 between 10' - 43' of water
2. P-4 between 9' - 35' of water
3. P-6 between 11' - of water

D. Prediction at failure:

1. P-3 between 12' - 46' of water
2. P-4 between 11' - 38' of water
3. P-6 between 13' of water

Additional figures and data used in Dr. Raymond's prediction are presented in Appendix F.

Professor Raymond's predictions are summarized in Table IV - 12.

C. P. Wroth

Dr. Wroth based his predictions on the Cam-clay model of clay behavior developed at Cambridge University.

Information on the Cam-clay model of soil behavior and details of the prediction are presented in Appendix G. Parameters for the Cam-clay model were selected from laboratory tests given in the symposium bulletins. Values used in the computations to model Boston Blue Clay were C_c = 0.338, Cr = 0.138, Γ = 2.15, ϕ' = 26.5° and ν = 0.35 (see Appendix G for notation.)

Behavior of the sand fill was based on a model developed by Simpson (1973) which incorporates provisions for strain softening and dilation. The existing model with parameters chosen to match Leighton Buzzard sand was used without modification.

The finite element program used for the predictions has many special features, the most notable being:

1. The plane strain elements used are simple 3-node constant strain triangles.
2. The program can handle strain-softening and dilation.
3. The program treats soil as a two-phase material; the behavior of the soil is always controlled by the effective stresses and equilibrium by total stresses. The pore-water is ascribed an appropriately large (but finite) bulk modulus, so that for the so-called undrained condition there has to be compatibility between the small volume change of the water and that of the soil skeleton. The program keeps

track of total and effective stresses for every element. For example, Fig. IV-28 shows the effective and total stress paths for the indicated element positioned under the center line of the embankment; the stage at which this element yielded is clearly shown by the abrupt change in the direction of the effective stress path at Y. At Z, the element has reached failure and is being held in equilibrium by the surrounding elements but unable to sustain any greater shear stress. The first part XY of the effective stress path is the partly drained simulation of the combined effects of the initial construction and subsequent five years of partial consolidation; the method is described below.

4. The program allows for the effect of independent rotation of the principal axes of stress, stress-increment and strain-increment.
5. The program includes a facility for refining the mesh of elements during a computation so that progressive failure and growth of failure zones may be modelled.

Matching Field Consolidation

The predictions are based on two separate computations. The first computation was modelling the construction of the initial embankment up to the level of +40 ft as a partially drained situation, for which a bulk modulus for the water K_w of 1×10^5 lbf/ft^2

was adopted; this gave a reasonable match to the field data available. The criterion for acceptability was when adequate agreement had been obtained for the computed horizontal movements with the field data from the inclinometers after 600 days (Aug. 1969). The computed profiles are given in Fig. IV-29.

The field data for the changes that occurred between 600 days (Aug. 1969) and 5 years (Aug. 1974) showed small changes in excess pore pressures but substantial settlements which were attributed mainly to undrained creep. Because the in situ effective stresses had not substantially altered during this period, it was believed that the computation represented as well as it could the existing stresses in the ground at the start of the test. The final stage was to model the filling of the embankment up to the predicted failure height of elevation +61. This was carried out as a completely undrained condition with the bulk modulus of water of 2×10^6 lbf/ft^2.

Failure

Failure of the embankment could not be properly modelled by the finite element program because the Cam-clay model reaches failure (as defined by maximum shear stress) for normally consolidated soil only after infinite strain.

Recourse was made to a stability analysis using the classical method of a slip circle. A limited search for the worst circle was made, and the parameters chosen for the shear strength were (1) fill material $c'=0$, $\phi'=30°$, $\gamma=120$ lbf/ft^3; (2) sand layer in the existing ground $c'=0$, $\phi'=35°$, $\gamma=120$ lbf/ft^3; (3) Boston Blue

Clay c_u = 8.8 lbf/in^2 (60 kN/m^2). The last figure was chosen partly on the quoted,in situ vane shear strengths, and partly on the results of undrained pressuremeter tests carried out at the site by Hughes using the Camkometer (self-boring pressuremeter) developed at Cambridge, Wroth and Hughes (1973).

No allowance was made for anisotropy of strength in the clay; a 10-ft vertical crack was assumed at the top of the fill where the slip circle reached the surface; reduced strength was taken for the small length of arc at the toe of the slip circle where it passed through the peat.

Dr. Wroth's predictions are summarized in Table IV-13.

R. H. Bassett

Dr. Bassett's prediction was based on a centrifuge model of the I-95 embankment. Appendix H presents a description of the centrifugal model technique.

The I-95 embankment model foundation material consisted of powdered Boston Blue Clay supplied by MIT and remixed with distilled water under vacuum at 1.7 to 1.9 times the liquid limit. The normal consolidation properties of this remolded material compared well with the reported properties. Study of Fig. 15 in the Symposium Bulletin suggested that a reasonable approximation to the prototype would be a two-layer model with OC clay with a P_c of 5.8 kip/ft^2 from -65 feet downward. The stiff clay from -12' to -18' elevation (P_c = 8.5 kip/ft^2) was omitted. The I-95 model had a simplified horizontal till strata.

Due to the limited capacity of the available centrifuge at Luton, England, a 1/180 scale model was tested at 60 g's (i.e., the model represented a prototype 1/3 the actual size). It was, therefore, necessary to consolidate materials at stresses one-third of those in the field. The slurry was K_o consolidated in two large containers (to a consolidation stress B_m = 0.62 ton/ft^2 for the NC clay (Layer B) and X_m = 0.84 ton/ft^2 for the OC clay (Layer A)) and then suitable layers were trimmed to the required thickness and placed in the test package. It was later found that the actual consolidation pressures were B_m = 0.73 ton/ft^2 and A_m = 1.15 ton/ft^2. The error in the lower

layer was very little but in the upper layer it was considerable.

The model was run in the centrifuge at 60 g's for 8 hours to bring it to stress similarity with a 1/3 full scale prototype. The machine was stopped and an embankment constructed to represent the original work. The model was again accelerated to represent construction as shown in Figures IV-30 and maintained for just over 2 1/2 hours to represent the various surcharge loading stages. During the whole sequence of operations, photographs for strain measurements and pore pressure transducer readings were taken.

Strain measurements for the construction of the original embankment show some indication of non-circular characteristics. They also give the first indication of multiple failure mechanisms acting together. At the end of the original embankment construction, it was known that the original laboratory consolidation had resulted in stiffer materials being used in the model than the ideal although the relative stiffnesses would be of the same ratio. In order to obtain a scale factor, the model and prototype performance were compared. The scale comparison between prototype and model for a reasonable fit on the horizontal displacement data would be 500 to 1 and for the vertical displacement 1000 to 1, not 180:1.

The embankment surcharge was contained in a hopper opened by a hydraulic piston which forms part of the test box. The hopper was actuated for three seconds about 130 minutes after

the original construction had been completed. The surcharge weight turned out to be equivalent to 8 feet of fill at 25% moisture content. This was reasonably close to the requested 6 foot value. The displacement results in terms of incremental values immediately after loading, are shown in Figures IV-31 and IV-32. They are shown as actual model displacements with no allowances for stiffness errors. The volumetric and shear strain are shown in Figures IV-33 and IV-34.

Finally, after a three-minute pause (equivalent to four weeks) to record pore water pressures, the hopper was opened for 10 seconds and the measurements repeated. The load was greater than intended and caused considerable widening of the base. The data for this last step is presented in Appendix H.

Dr. Bassett's predictions are summarized in Table IV-14. The displacement predictions were obtained by multiplying the actual model displacements by the scaling factor obtained before from comparison with actual field data.

TABLE IV-1. SUMMARY OF SOIL PARAMETERS FOR DAVIS AND POULOS ANALYSIS

SOIL TYPE	PARAMETER	CORRELATION	RANGE OF VALUE	BASIS
BOSTON BLUE CLAY	OCR	---	---	Lab and Field Data
	S_u	$\frac{S_u}{\bar{\sigma}_{vo}}$ vs. OCR	$\frac{S_u}{\bar{\sigma}_{vo}}$ = 0.23, OCR = 1 = 0.55, = 3 = 0.94, = 7	Lab test results and published data
	E_u	$\frac{E_u}{\bar{\sigma}_{vo}}$ vs. OCR	$\frac{E_u}{\bar{\sigma}_{vo}}$ = 67, OCR = 1 = 116, = 3 = 183, = 7	Lab test results published data and adjustment in Stage 1
	$\bar{E}$	$\bar{E}$ vs. depth	$\bar{E}$ = 25 kg/cm^2, el. -25 ft. = 10 kg/cm^2, el. -105 ft.	Published data
	$\bar{\nu}$	$\bar{\nu}$ vs. depth	$\bar{\nu}$ = 0.3, el. -25 ft. = 0.45, el. -105 ft.	Published data
	C_v	Normally consolidated	15 x 10^{-4} cm^2/sec	
		Overconsolidated	45 x 10^{-4} cm^2/sec	Published data
	C_h/C_v	---	7	---
SAND LAYER AND FILL	$\bar{\phi}$	---	35°	Lab test results
	K_o	---	0.8	and judgment
	$\bar{E}$	$\bar{E}/\bar{\sigma}_{vo}$	$\bar{E}/\bar{\sigma}_{vo}$ = 200	
	$\bar{\nu}$	---	0.4	
TILL	Permeability	---	Effectively impermeable	Stage 1 analysis

TABLE IV-2. SUMMARY OF PREDICTIONS

Poulos & Davis

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	27.0 ft
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	0.17" (All eastwards)
El. -70	0.016" (All eastwards)
SI-4 El. -30	0.47" (All eastwards)
El. -70	0 (All eastwards)
Additional settlement of SP-1 for 6 ft extra fill	0.76"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	8.8
P4	9.5
P6	8.1
Additional heave for 6 ft extra fill	
H1	0.23"
H2	0.32"
Additional pore pressures at failure (in ft of water)	
P3	17.9
P4	22.1
P6	12.1

TABLE IV-3. W.W. MOORE - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	13 ft
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	0.75" All eastwards
El. -70	0.75"
SI-4 El. -30	0.75"
El. -70	0.50"
Additional settlement of SP-1 for 6 ft extra fill	1.25"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	8.8
P4	7.2
P6	4.8
Additional heave for 6 ft extra fill	
H1	1.0"
H2	0.5"
Additional pore pressures at failure (in ft of water)	
P3	16.6
P4	16.6
P6	11.9

TABLE IV-4. PREDICTED EXCESS PORE PRESSURES STAGE III, I-95 STATION 263

PIEZ. No.	EL. (ft) OFFSET	OCR	YIELD RATIO EL.40	YIELD RATIO EL.46	$\Delta\sigma_{oct}$ (psf)	$\Delta\tau_{oct}$ (psf)	a	$\Delta\sigma_{oct} + a\,\Delta\tau_{oct}$ psf	$\Delta\sigma_{oct} + a\,\Delta\tau_{oct}$ ft. H_2O	PREDICTED Δu ft WATER
	-35.5						1.0	660	10.5	14
P-3		1.44	0.77	0.82	505	155	2.5	895	14.3	
	℄						4.0	1125	18.0	(10 - 18)
	-57.5						5	740	11.8	14
P-4		1.05	0.96	0.98	590	30	10	890	14.3	
	℄						15	1040	16.7	(12 - 17)
	-40.5						0	500	8.0	10
P-6		2.35	0.49	0.64	500	183				
	115'E						1.5	775	12.4	(8 - 12)

Note: $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$ computed from FEECON 2

Ladd

TABLE IV-5. C.C. LADD - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	23.0 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	0.7"
El. -70	0.35"
SI-4 El. -30	0.45"
El. -70	0
	All eastwards
Additional settlement of SP-1 for 6 ft. extra fill	0.8"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	14
P4	14
P6	10
Additional heave for 6 ft. extra fill	
H1	0.5"
H2	0.4"
Additional pore pressures at failure (in ft. of water)	
P3	37
P4	39
P6	25

TABLE IV-6. Z.C. MOH - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	9 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	1.5" (All eastwards)
SI-3 El. -70	1.25" (All eastwards)
SI-4 El. -30	1.0" (All eastwards)
SI-4 El. -70	0 " (All eastwards)
Additional settlement of SP-1 for 6 ft. extra fill	3.0"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	9.5
P4	9.2
P6	11.5
Additional heave for 6 ft. extra fill	
H1	0.25"
H2	1.25"
Additional pore pressures at failure (in ft. of water)	
P3	12.8
P4	12.5
P6	17.1

TABLE IV-7. EFFECT OF 6 FEET OF ADDITIONAL FILL ON SP-1, SI-3 & SI-4 EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS

DEFORMATIONS (NO STRENGTH GAIN)

	LAST 6 FEET	LOAD FACTOR (39/33)	RATE FACTOR (9 TO 1)	ADDITIONAL 6 FEET
SI-3 (-30')	1.1"	1.18	0.9	1"
SI-3 (-70')	0.9"	1.18	0.9	1"
SI-4 (-30')	0.3"	1.18	0.9	1/4"
SI-4 (-70')	0"	1.18	0.9	0"
SP-1	3.8"	1.18	0.9	3 1/2"

A. Casagrande & S.D. Wilson, 1950, Sudden Deformations were Found to be Proportional to the Magnitude of the Load and Are Little Affected by Small Changes in Rates of Loading.

TABLE IV-8. PORE PRESSURE VS FILL HEIGHT EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS

CONSTRUCTION PERIOD	$\Delta\mu$ PIEZOMETER "B" (P.S.F.)	ΔH (FILL) (FEET)	ΔP (γ=119) (P.S.F.)	ΔP (γ=129) (P.S.F.)
1	750	9 (Equiv)	1070	1150
2	3375	24	2860	3080
3	870	6	720	770
Total	5000	39	4650	5000

TABLE IV-9. McGUFFEY & GEMME - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	13.5 ft
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	.25" (All eastwards)
SI-3 El. -70	.25" (All eastwards)
SI-4 El. -30	.25" (All eastwards)
SI-4 El. -70	0 (All eastwards)
Additional settlement of SP-1 for 6 ft. extra fill	3.0"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	7.3
P4	7.2
P6	4.9
Additional heave for 6 ft. extra fill	
H1	1.0"
H2	1.0"
Additional pore pressures at failure (in ft. of water)	
P3	14.8
P4	13.9
P6	7.6

TABLE IV-10. J.M. DUNCAN - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	12 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	1.2" (All eastwards)
SI-3 El. -70	0.6" (All eastwards)
SI-4 El. -30	0.6" (All eastwards)
SI-4 El. -70	0 (All eastwards)
Additional settlement of SP-1 for 6 ft. extra fill	1.8"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	19.22
P4	20.82
P6	12.81
Additional heave for 6 ft. extra fill	
H1	0.6"
H2	1.2"
Additional pore pressures at failure (in ft. of water)	
P3	38.44
P4	41.65
P6	25.63

TABLE IV-11. R.C. KIRBY - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	23.5 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	-1.2" } All
El. -70	-0.8" } eastwards
SI-4 El. -30	-0.6" }
El. -70	
Additional settlement of SP-1 for 6 ft. extra fill	-5"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	12 ft.
P4	12 ft.
P6	2.5 ft.
Additional heave for 6 ft. extra fill	
H1	No Prediction
H2	
Additional pore pressures at failure (in ft. of water.)	
P3	25.5 ft.
P4	25.5 ft.
P6	No Prediction

TABLE IV-12. G.P. RAYMOND - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	8 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	8.6" } All eastwards
SI-3 El. -70	8.6" } All eastwards
SI-4 El. -30	5.3" } All eastwards
SI-4 El. -70	0 } All eastwards
Additional settlement of SP-1 for 6 ft. extra fill	13.7"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	26
P4	22
P6	11
Additional heave for 6 ft. extra fill	
H1	4.8"
H2	6.2"
Additional pore pressures at failure (in ft. of water)	
P3	29
P4	24
P6	13

TABLE IV-13. C.P. WROTH - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	21 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	1.6" (All eastwards)
SI-3 El. -70	0.6" (All eastwards)
SI-4 El. -30	0.7" (All eastwards)
SI-4 El. -70	0 (All eastwards)
Additional settlement of SP-1 for 6 ft. extra fill	2.6"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	9
P4	9.6
P6	5.8
Additional heave for 6 ft. extra fill	
H1	0
H2	0
Additional pore pressures at failure (in ft. of water)	
P3	27
P4	28.3
P6	7.9

TABLE IV-14. R.H. BASSETT - PREDICTIONS

QUANTITY	PREDICTED VALUE
Additional height of fill to cause stability failure	13 ft.
Additional horizontal movements for 6 ft. extra fill	
SI-3 El. -30	0.6" } All eastwards
SI-3 El. -70	2.0" } All eastwards
SI-4 El. -30	2.5" } All eastwards
SI-4 El. -70	2.25" } All eastwards
Additional settlement of SP-1 for 6 ft extra fill	4.75"
Additional pore pressures (in ft. of water) for 6 ft. extra fill	
P3	6
P4	8
P6	4
Additional heave for 6 ft. extra fill	
H1	2"
H2	4"
Additional pore pressures at failure (in ft. of water)	
P3	LOST
P4	15
P6	4

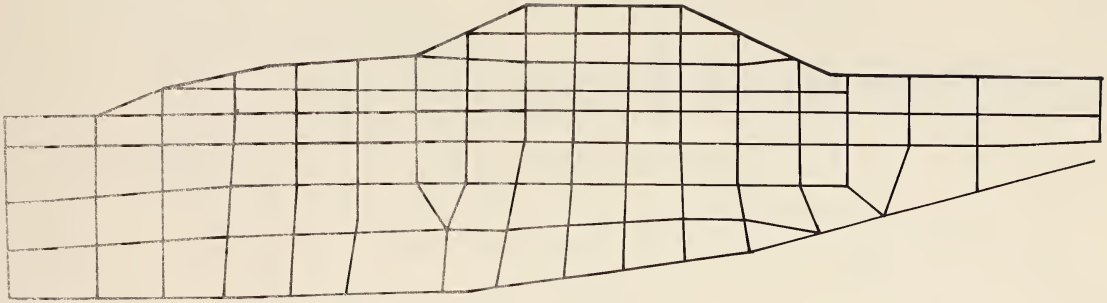


FIGURE 4-1. FINITE ELEMENT REPRESENTATION OF PROBLEM
POULOS AND DAVIS

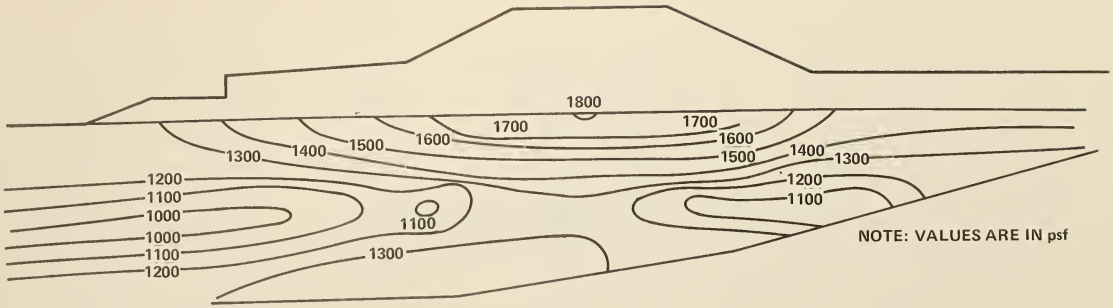


FIGURE 4-2. CONTOURS OF CLAY S_u VALUES USED FOR PREDICTIONS

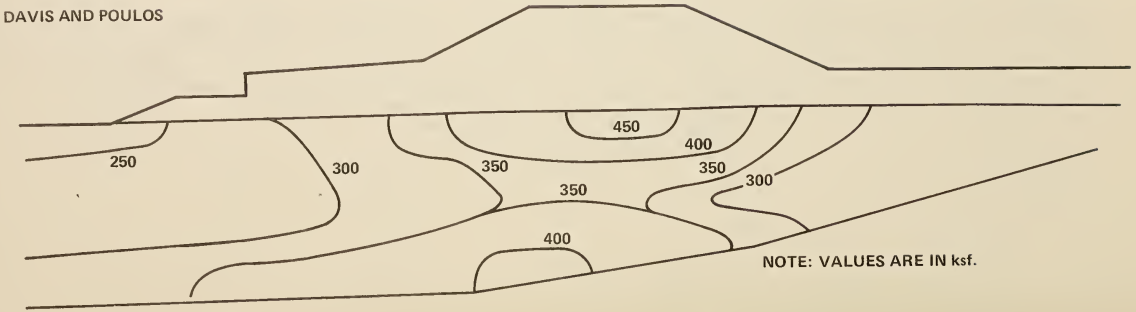


FIGURE 4-3. CONTOURS OF CLAY E_u VALUES USED FOR PREDICTIONS

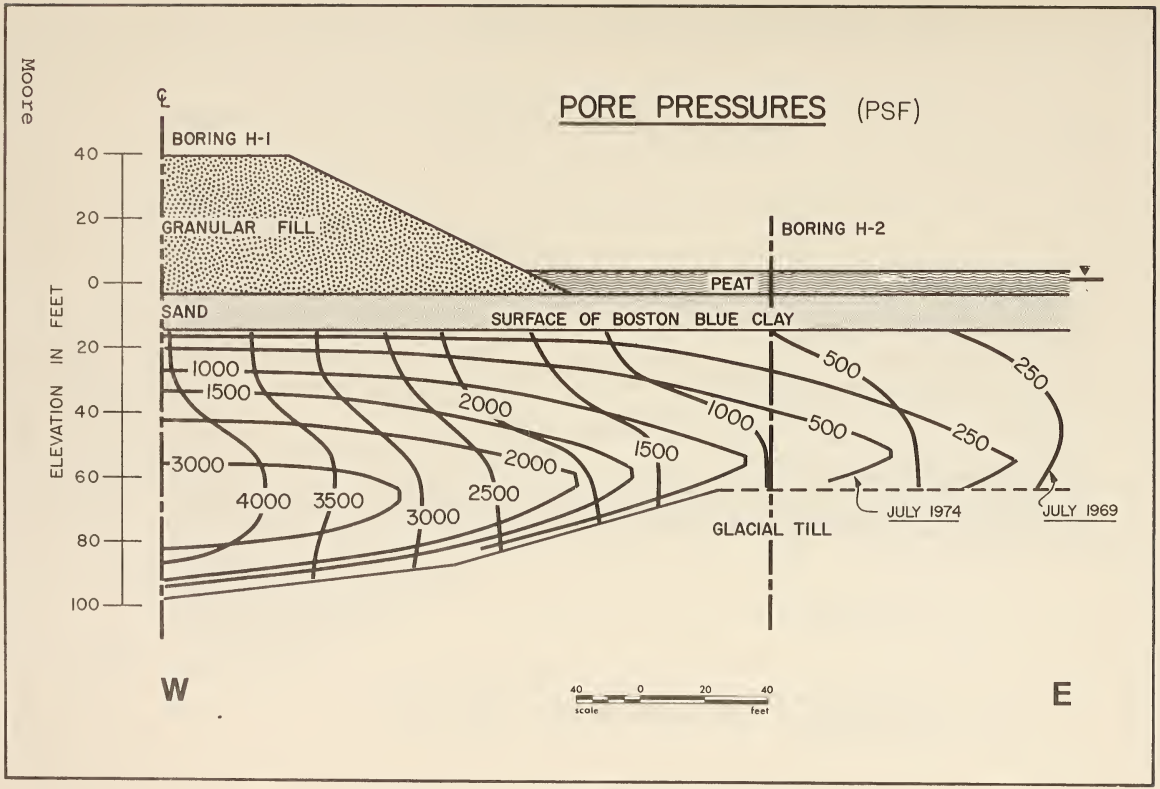


FIGURE 4-4. PORE PRESSURES (PSF)

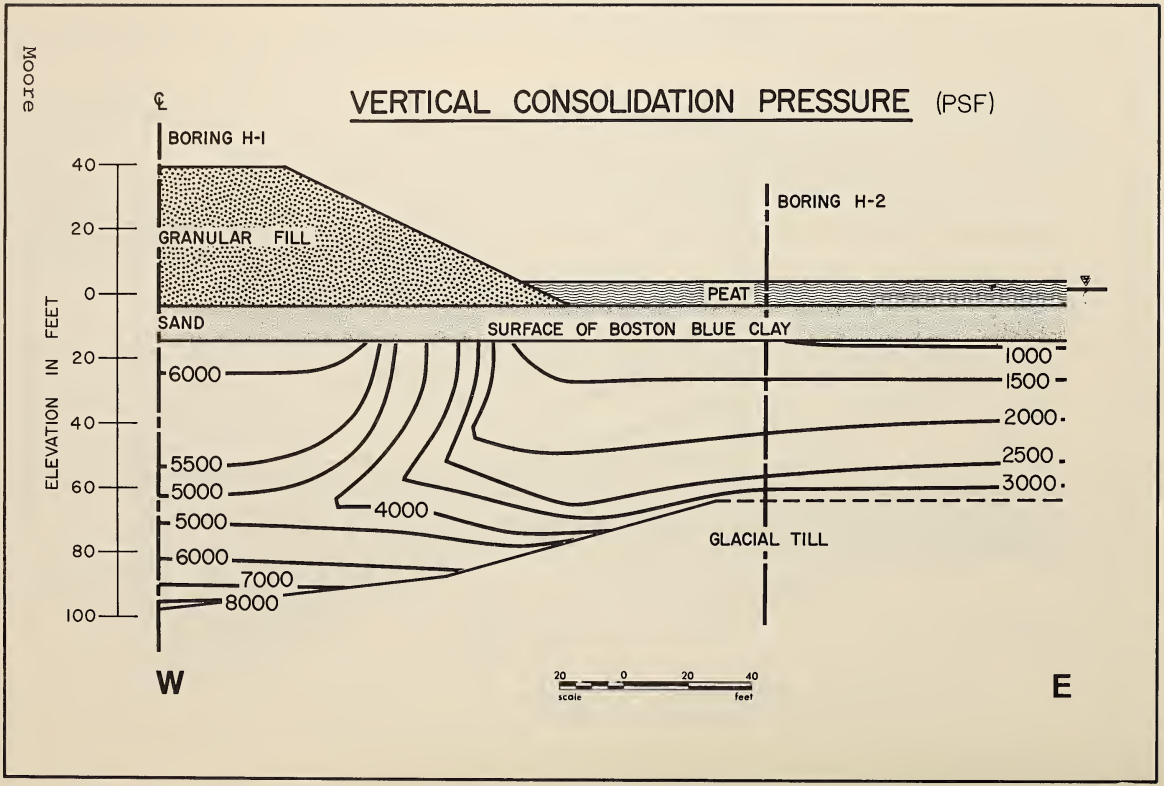


FIGURE 4-5. VERTICAL CONSOLIDATION PRESSURE (PSF)

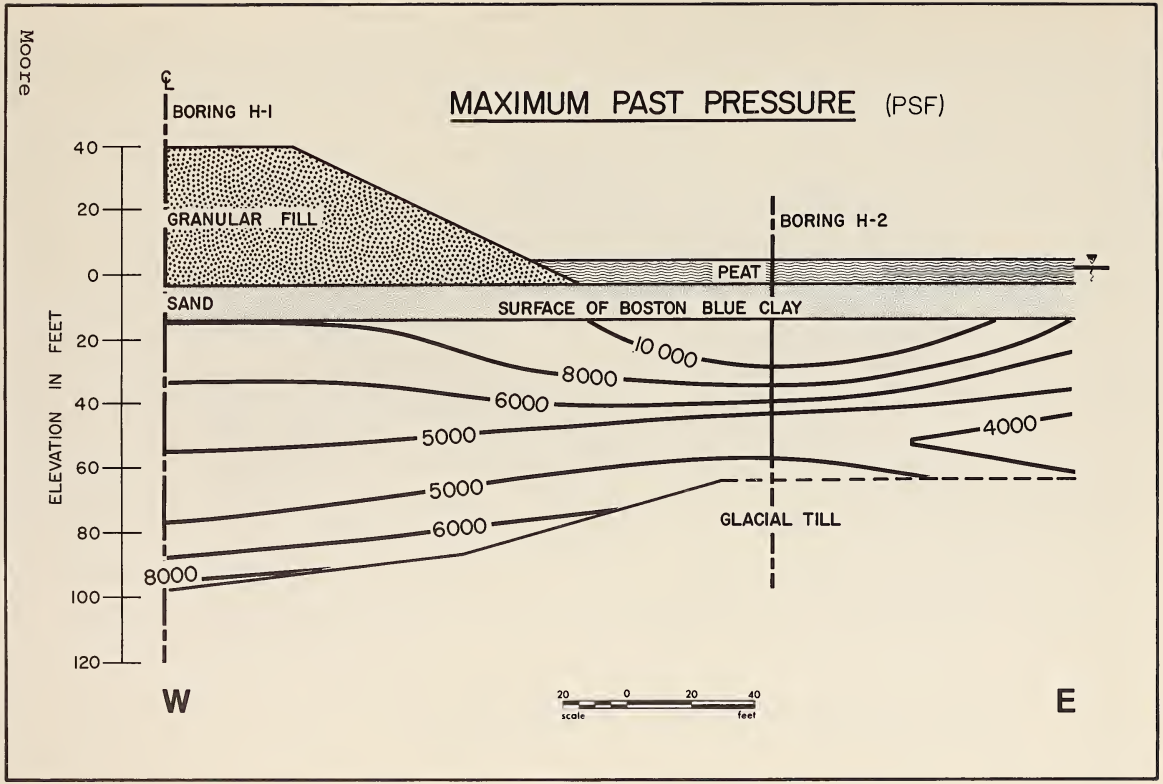


FIGURE 4-6. MAXIMUM PAST PRESSURE (PSF)

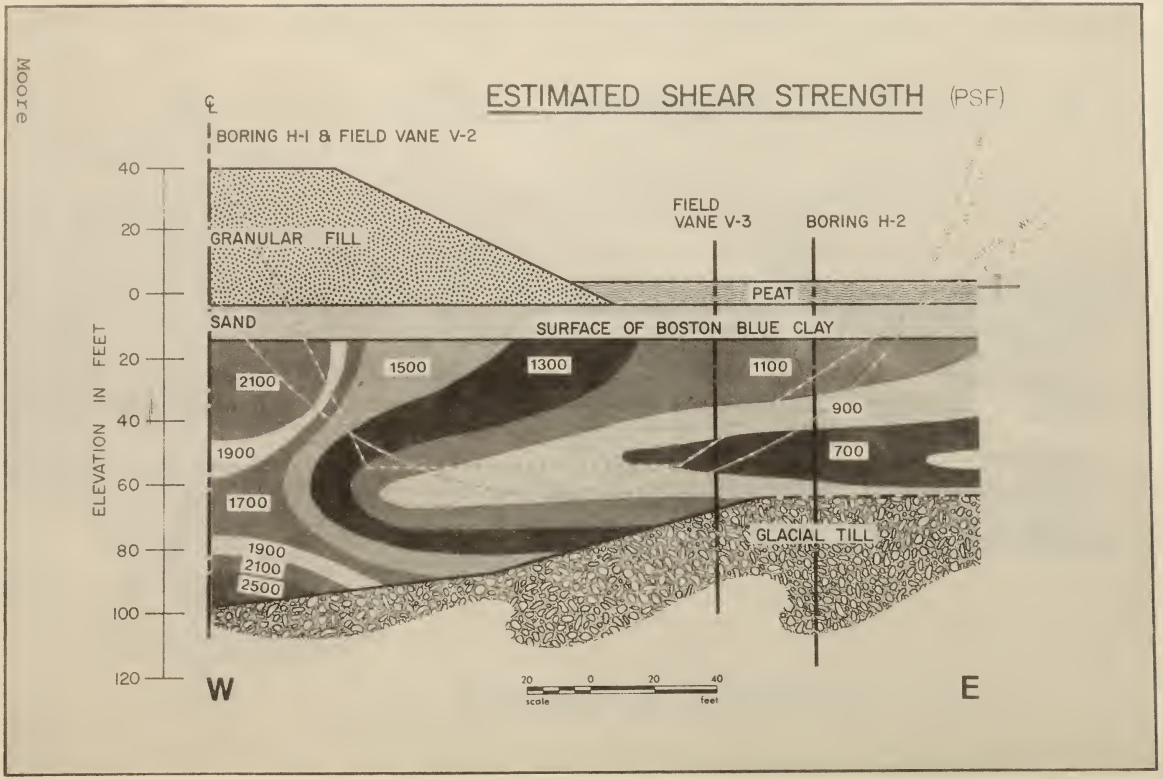
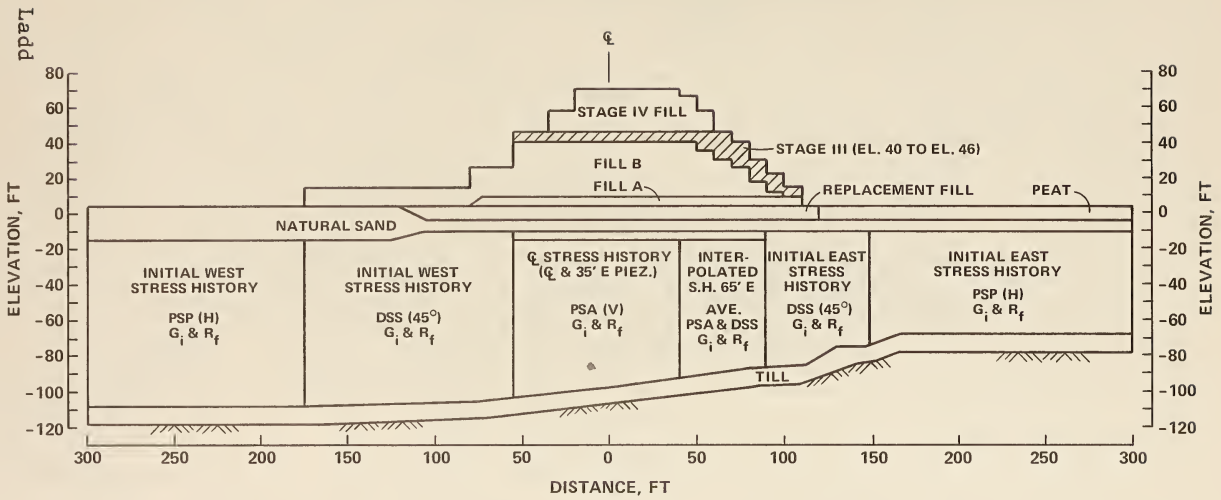


FIGURE 4-7. ESTIMATED SHEAR STRENGTH (PSF)



NOTE: Stress History ($\overline{\sigma}_{vc}$, $\overline{\sigma}_{vm}$ & OCR) Used to Compute S_u (β) and Modulus but Input Initial (Preconstruction) Values of $\overline{\sigma}_{vo}$ and K_o

FIGURE 4-8. SOILS FOR FEECON 2 ANALYSIS I-95 STATION 263

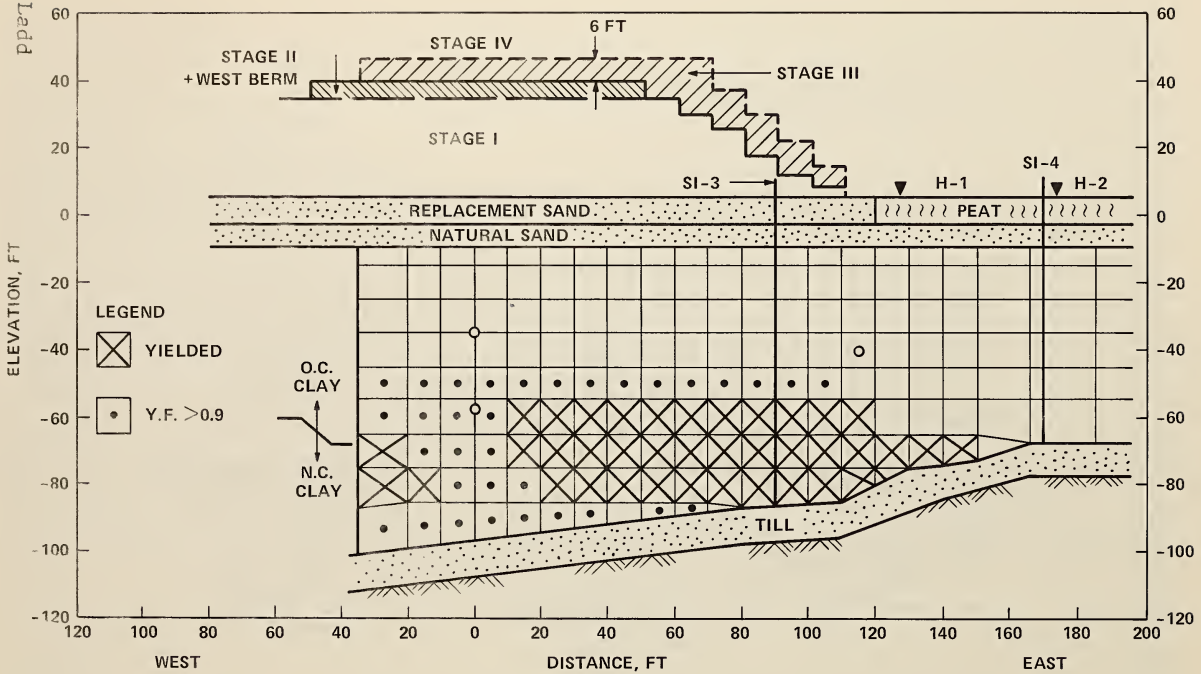
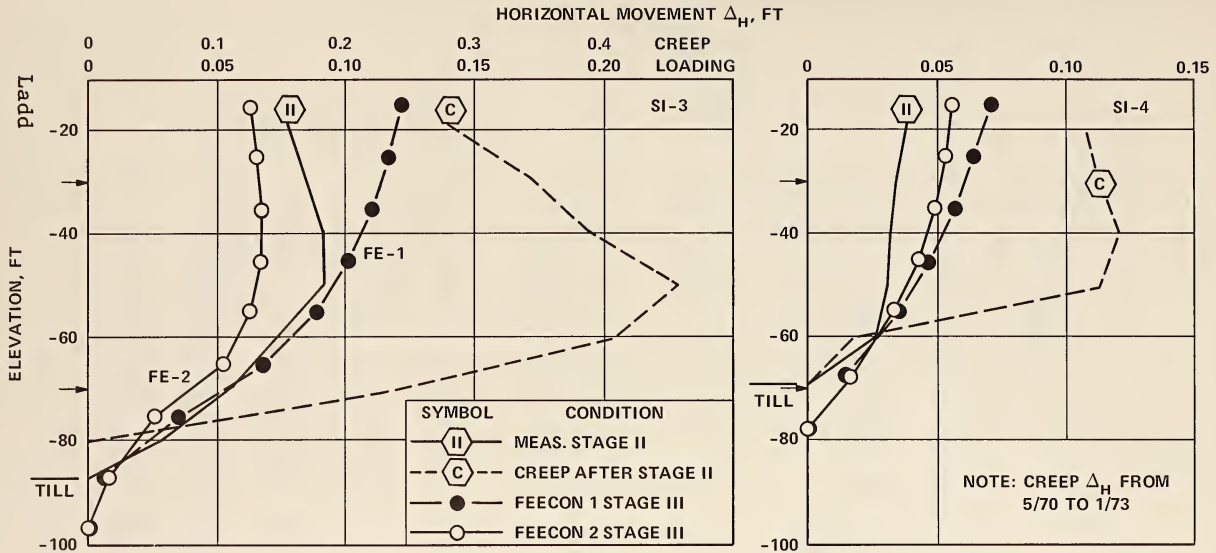


FIGURE 4-9. YIELDED AND HIGHLY STRESSED ZONES FEECON 2 STAGE III EL. 46
I-95 STATION 263 (+6' FILL & SIDE SLOPE)



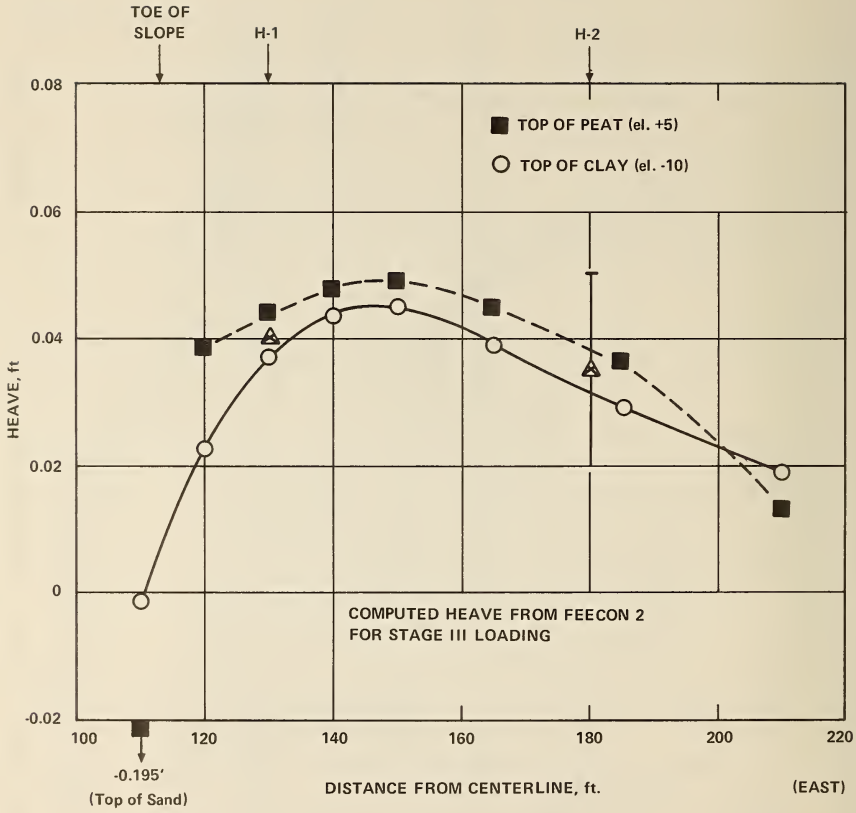
EL. - 30 Δ_H = FE2 - Δ_H (TILL) = 0.060′ ≈ 0.7″
RANGE = 0.5 - 0.9″

EL. - 70 Δ_H = FE2 - Δ_H (TILL) = 0.033′ ≈ 0.35″
RANGE = 0.25 - 0.45″

EL. - 30 Δ_H = FE2 - Δ_H (TILL) = 0.037′ ≈ 0.45″
RANGE = 0.3 - 0.6″

EL. - 70 Δ_H = 0

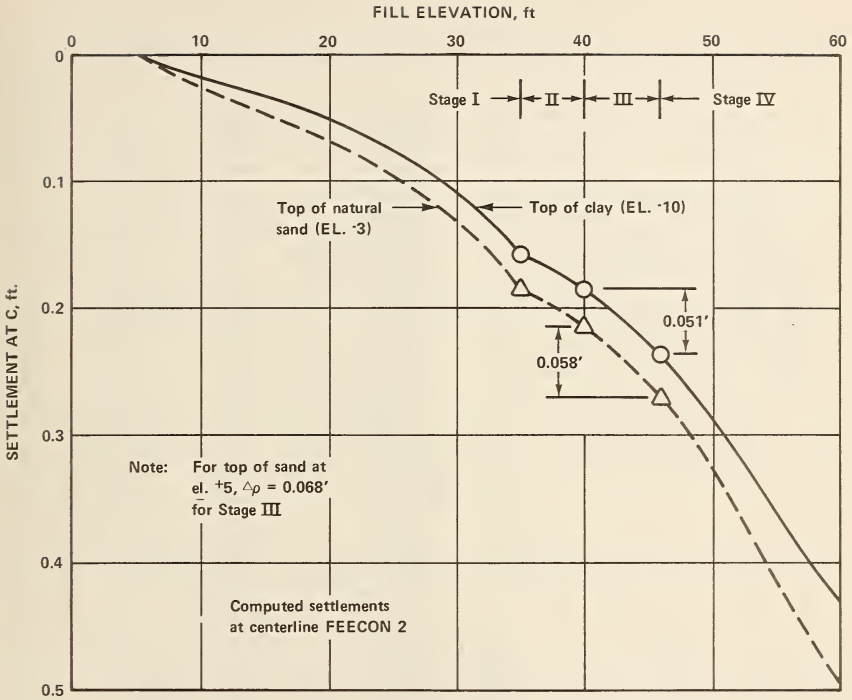
FIGURE 4-10. PREDICTED LATERAL DEFORMATIONS STAGE III, I-95 STATION 263



HEAVE POINT	BEST ESTIMATE	RANGE
H-1	0.04'	Filling may cause toe failure with very large heave (1-2ft)
H-2	0.035'	0.02 - 0.05'

FIGURE 4-11. PREDICTED HEAVE AT H-1 AND H-2 FROM STAGE III LOADING I-95 STATION 263

Ladd



FROM STAGE II FIELD DATA (Loading t = 32 days)

$\rho_t = \rho_i + \rho_c = 0.19'$, Est. $\rho_i = 0.10'$, $\therefore \rho_c = 0.09'$

For 6' fill with loading t = 5 days

Say $\rho_c = 0.01$ 0.005'

ESTIMATED SETTLEMENT AT CENTERLINE STAGE III

Maximum	ρ_i (ft)	ρ_c (ft)	ρ_t (ft)
Maximum	0.068	0.015	0.083 ~ 0.10
Best estimate	0.058	0.010	0.068 ~ 0.07 ←
Minimum	0.051	0.005	0.056 ~ 0.05

FIGURE 4-12. PREDICTED CENTERLINE SETTLEMENT, STAGE III I-95 STATION 263

Ladd

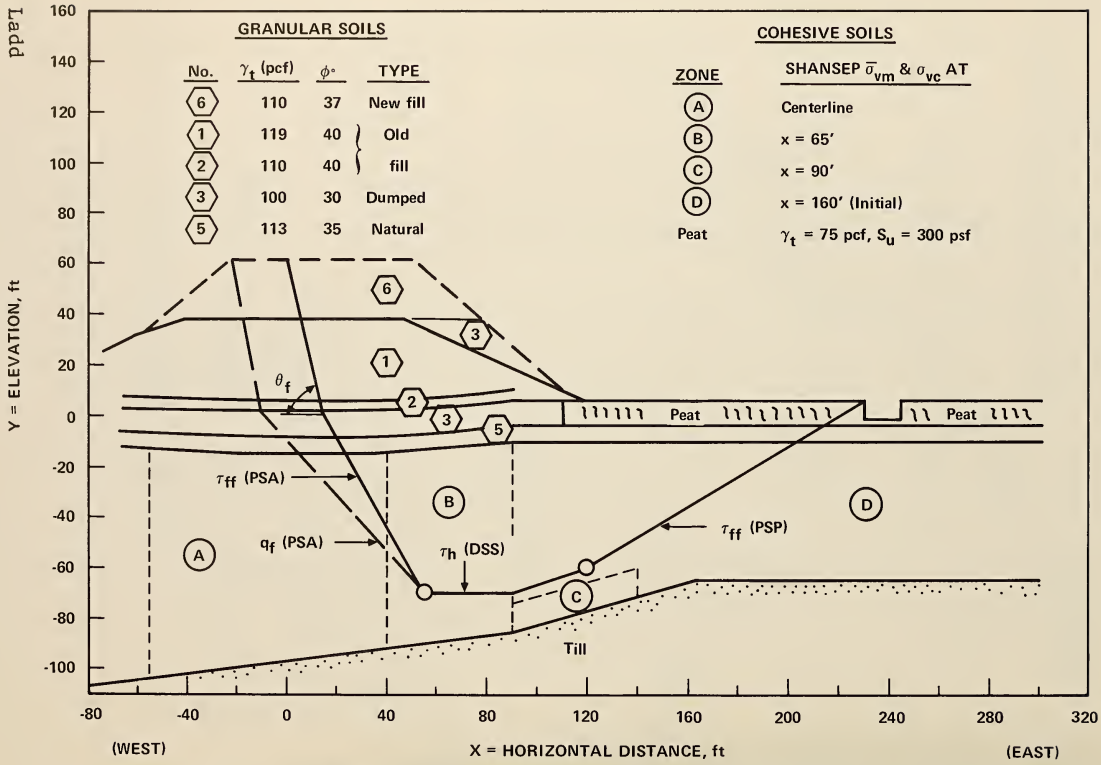


FIGURE 4-13. STRENGTH PARAMETERS FOR M-P STABILITY ANALYSES, I-95 STATION 263

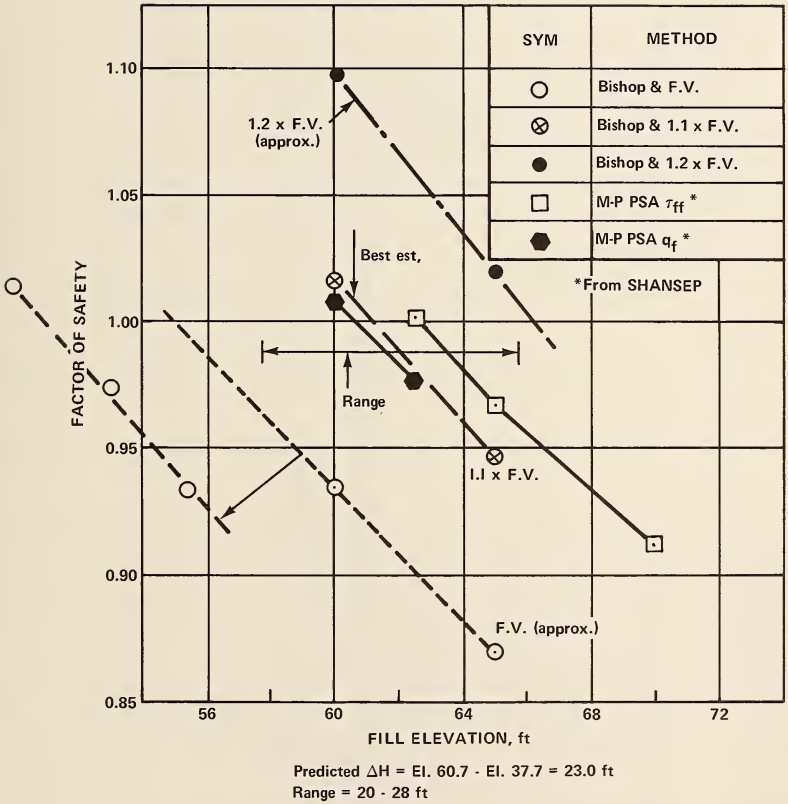


FIGURE 4-14. FACTOR OF SAFETY VS. FILL ELEVATION AND PREDICTED ΔH CAUSING FAILURE, I-95 STATION 263

Ladd

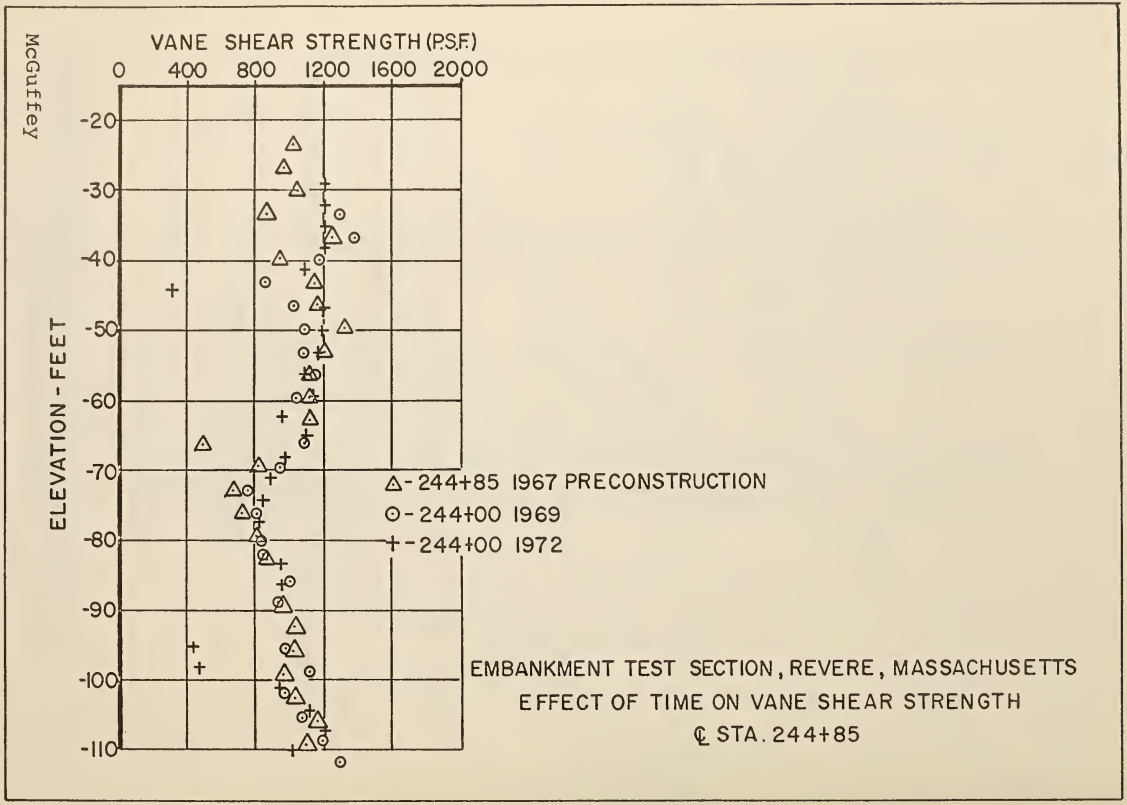


FIGURE 4-15. EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS EFFECT OF TIME ON VANE SHEAR STRENGTH ℄ STA. 244+85

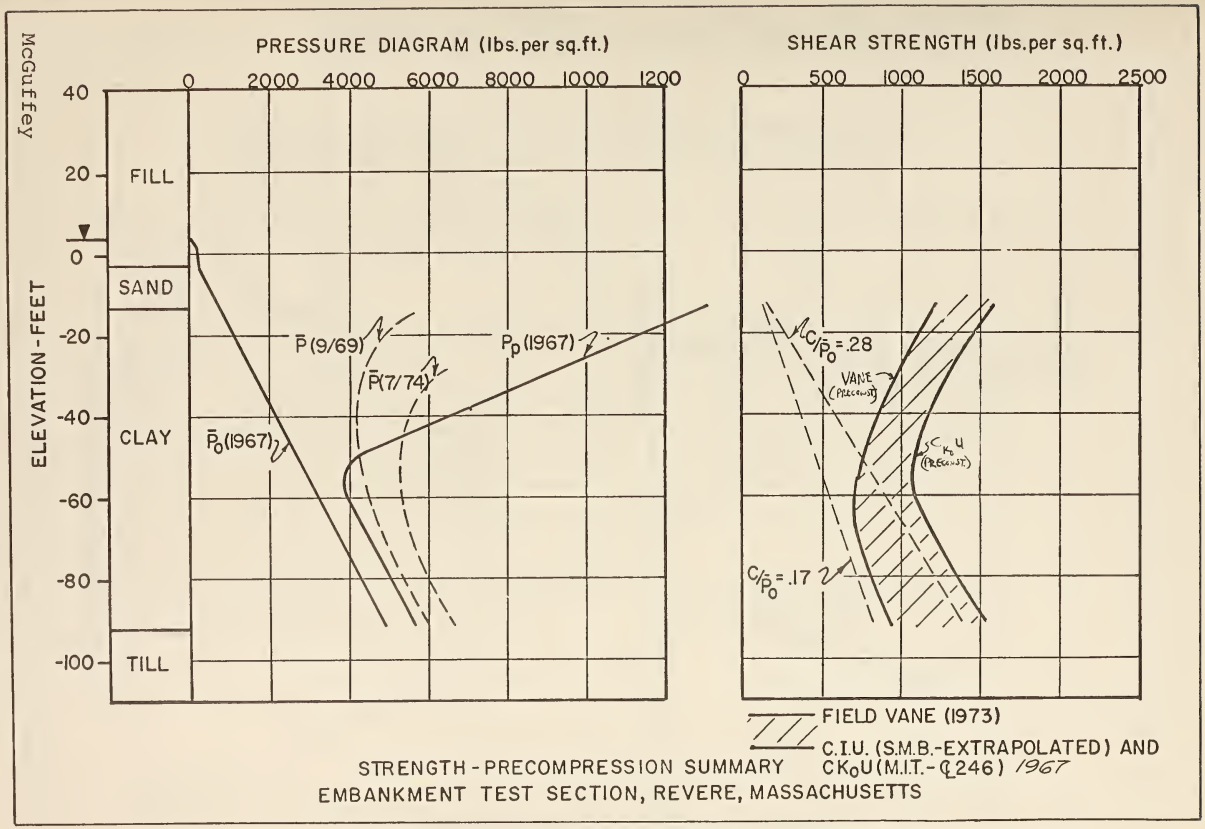


FIGURE 4-16. STRENGTH-PRECOMPRESSION SUMMARY EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS

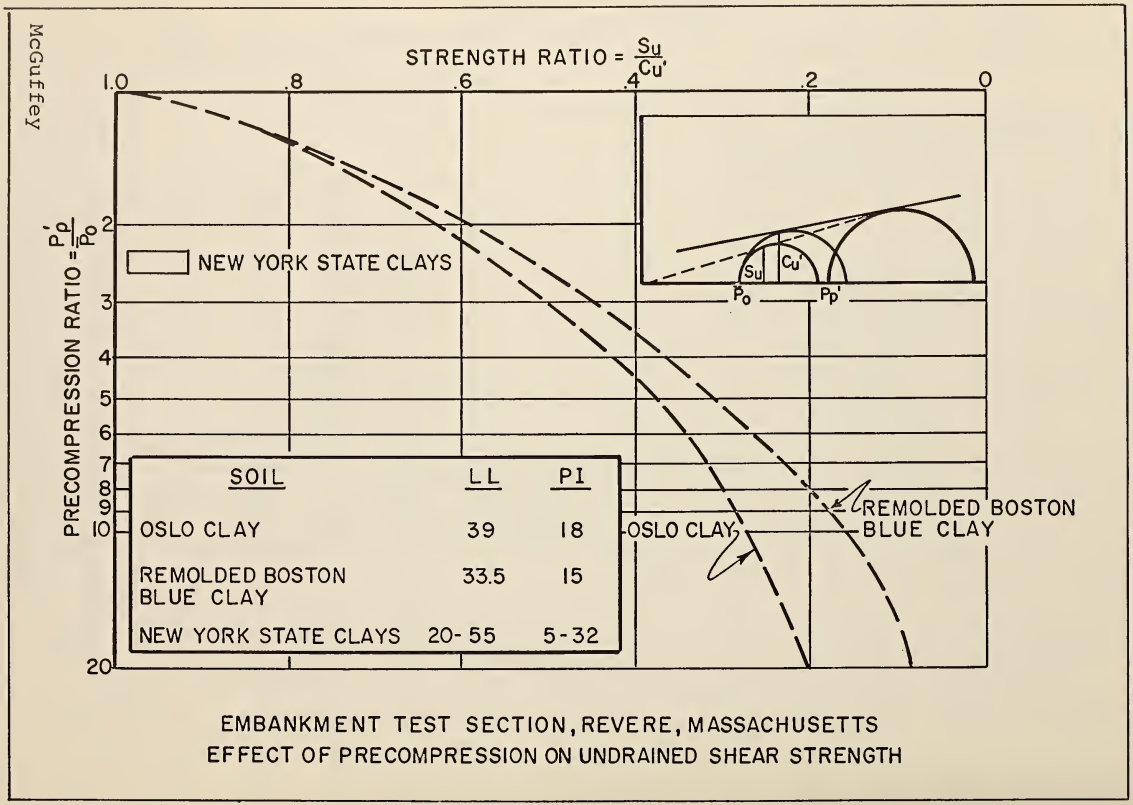


FIGURE 4-17. EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS EFFECT OF EFFECT OF PRECOMPRESSION ON UNDRAINED SHEAR STRENGTH

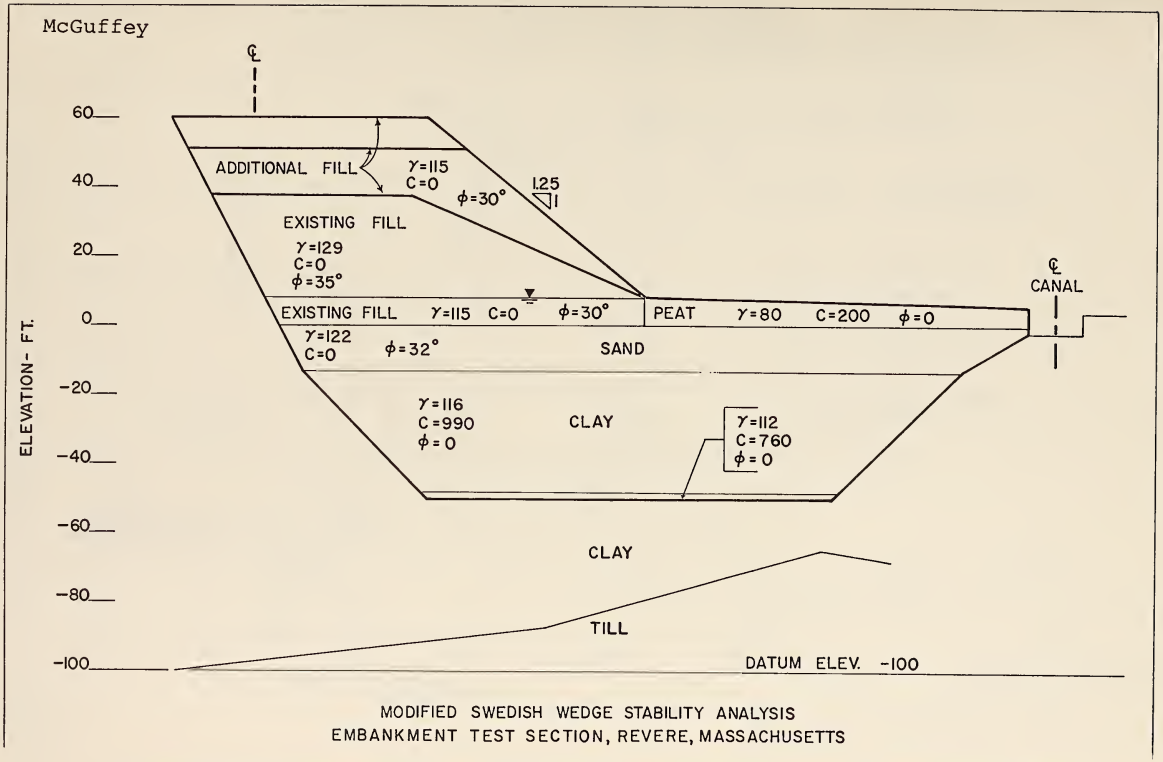


FIGURE 4-18. MODIFIED SWEDISH WEDGE STABILITY ANALYSIS EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS

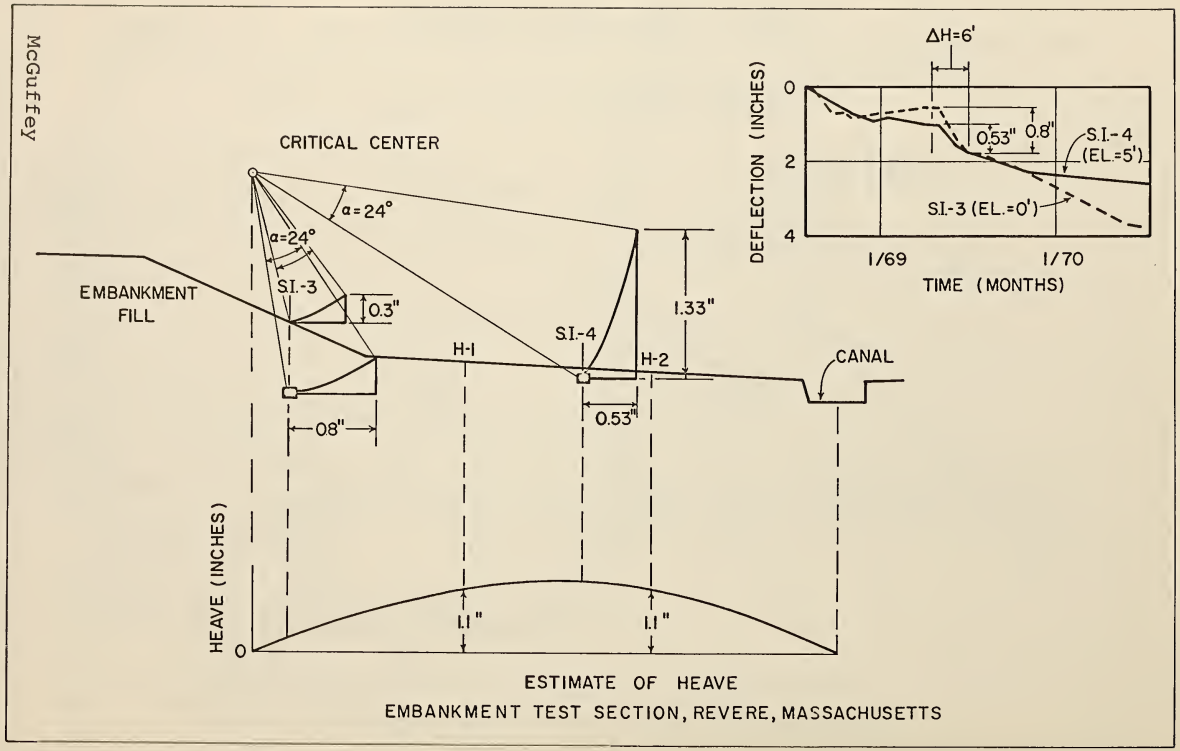
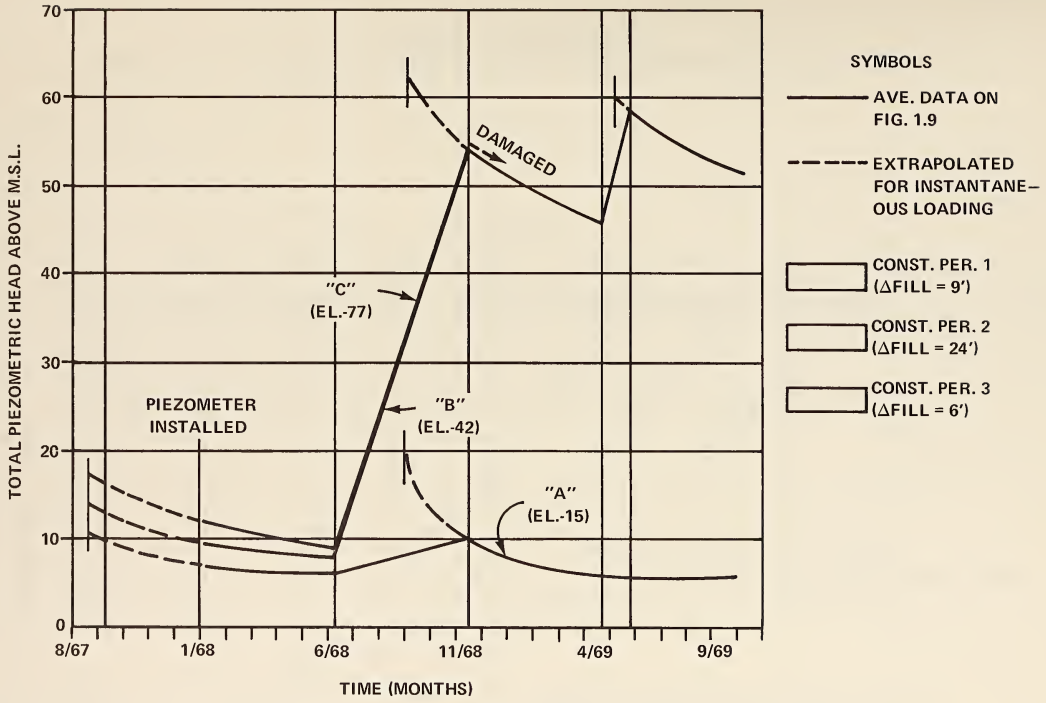


FIGURE 4-19. ESTIMATE OF HEAVE EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS



McGuffey

FIGURE 4-20. EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS
PIEZOMETER DATA

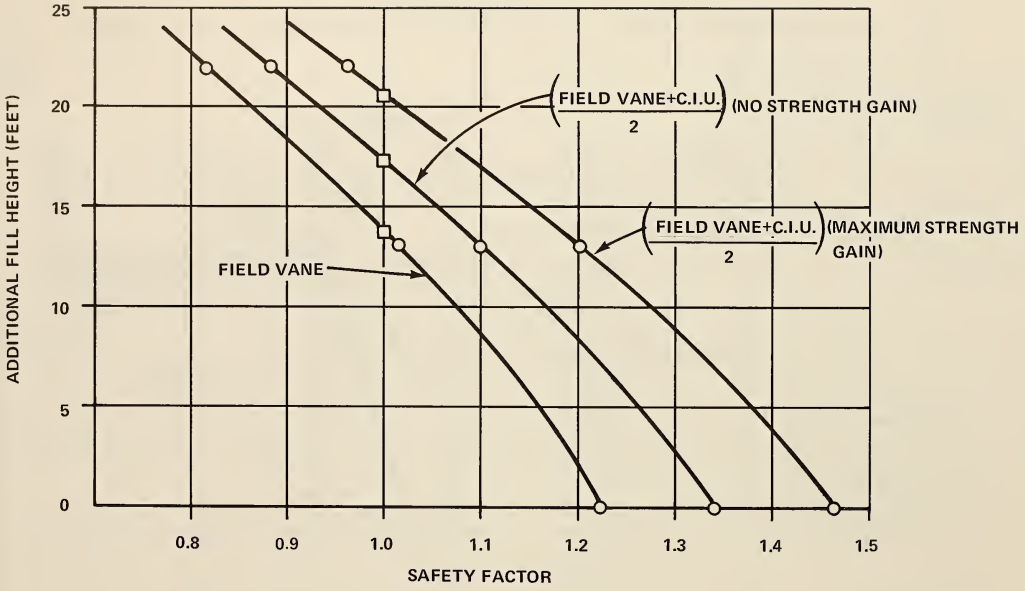


FIGURE 4-21. SAFETY FACTOR VS FILL HEIGHT EMBANKMENT TEST SECTION, REVERE, MASSACHUSETTS

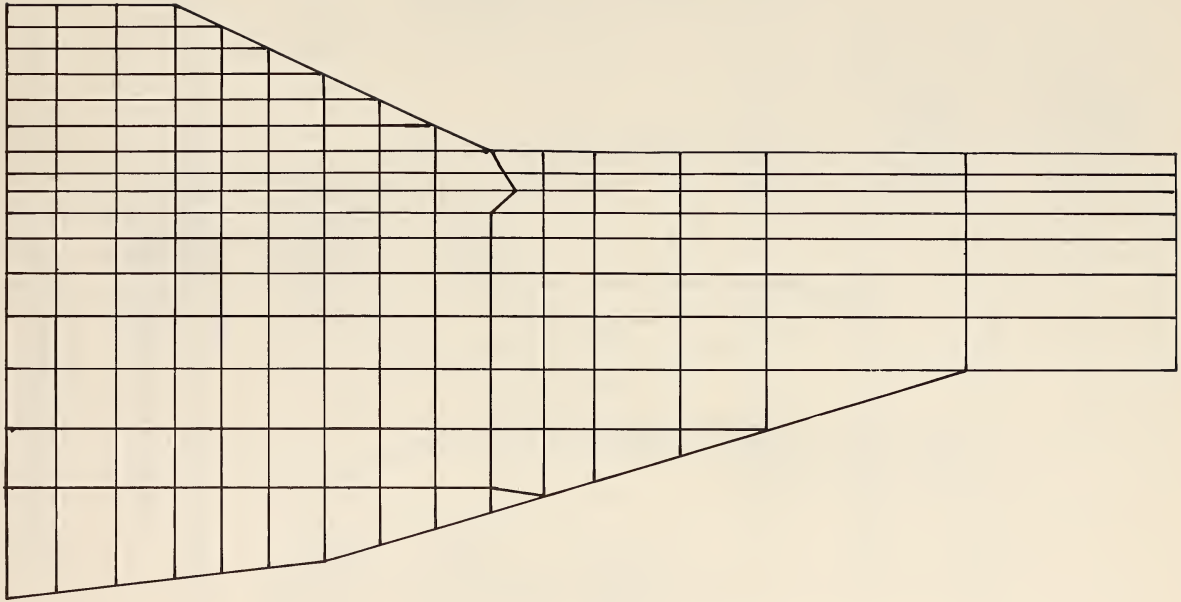


FIGURE 4-22. FINITE ELEMENT GRID - DUNCAN

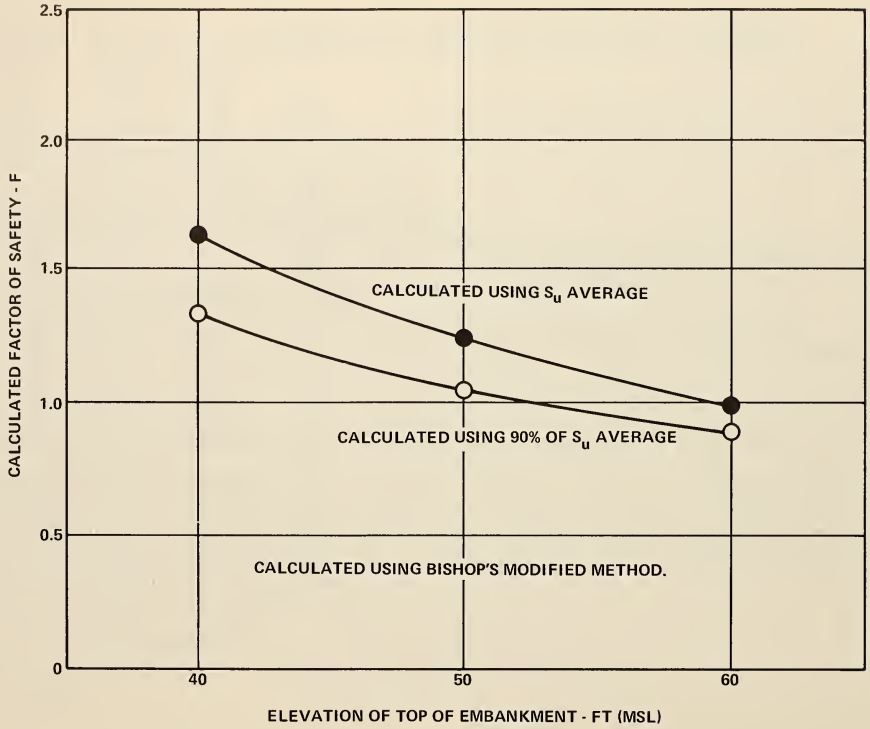


FIGURE 4-23. VARIATION OF FACTOR OF SAFETY WITH EMBANKMENT HEIGHT

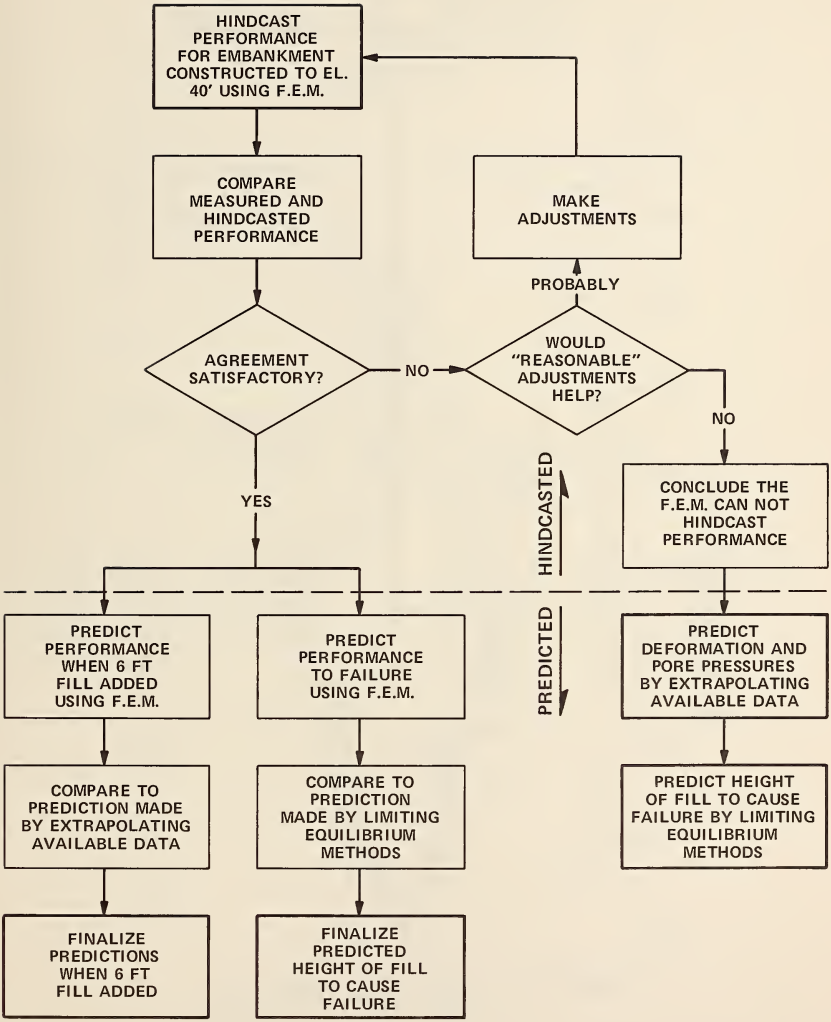

HINDCAST PERFORMANCE FOR EMBANKMENT CONSTRUCTED TO EL. 40' USING F.E.M.
COMPARE MEASURED AND HINDCASTED PERFORMANCE
MAKE ADJUSTMENTS
PROBABLY
AGREEMENT SATISFACTORY?
NO
WOULD "REASONABLE" ADJUSTMENTS HELP?
NO
YES
HINDCASTED
CONCLUDE THE F.E.M. CAN NOT HINDCAST PERFORMANCE
PREDICTED
PREDICT PERFORMANCE WHEN 6 FT FILL ADDED USING F.E.M.
PREDICT PERFORMANCE TO FAILURE USING F.E.M.
PREDICT DEFORMATION AND PORE PRESSURES BY EXTRAPOLATING AVAILABLE DATA
COMPARE TO PREDICTION MADE BY EXTRAPOLATING AVAILABLE DATA
COMPARE TO PREDICTION MADE BY LIMITING EQUILIBRIUM METHODS
PREDICT HEIGHT OF FILL TO CAUSE FAILURE BY LIMITING EQUILIBRIUM METHODS
FINALIZE PREDICTIONS WHEN 6 FT FILL ADDED
FINALIZE PREDICTED HEIGHT OF FILL TO CAUSE FAILURE

FIGURE 4-24. OUTLINE OF PROBLEM APPROACH

Kirby

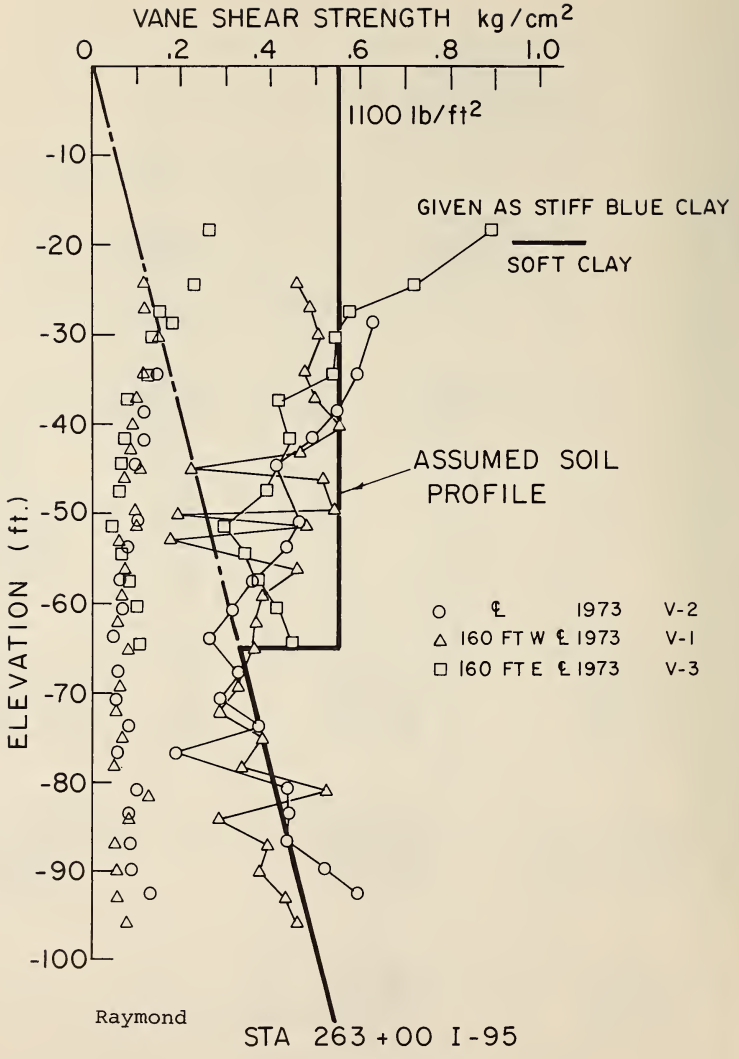
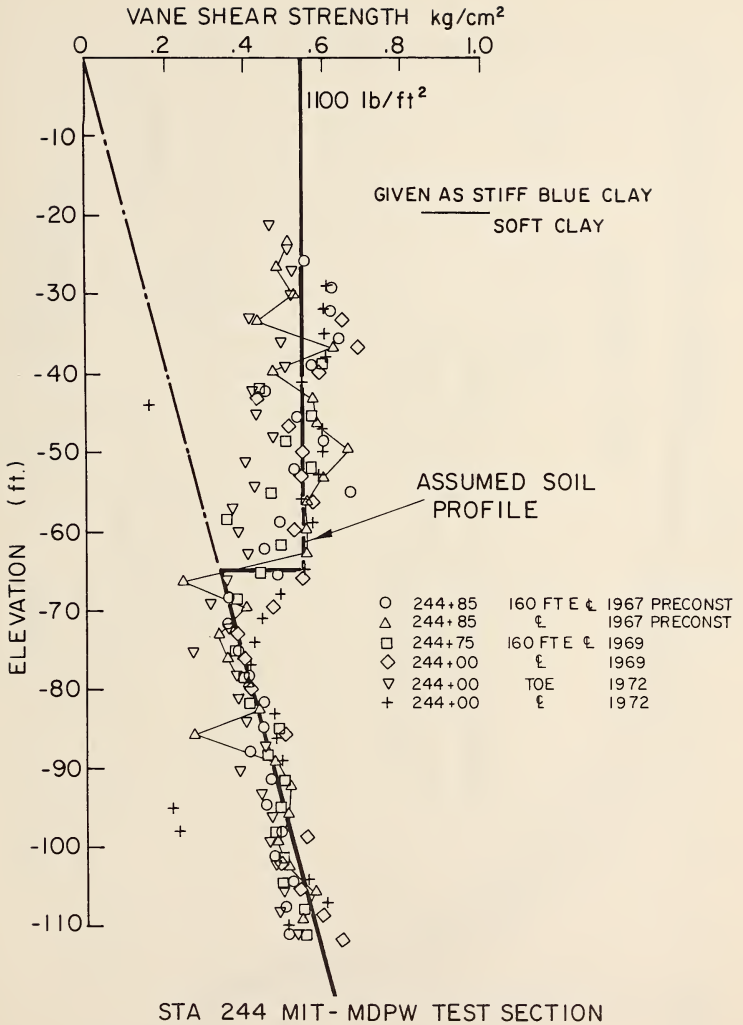
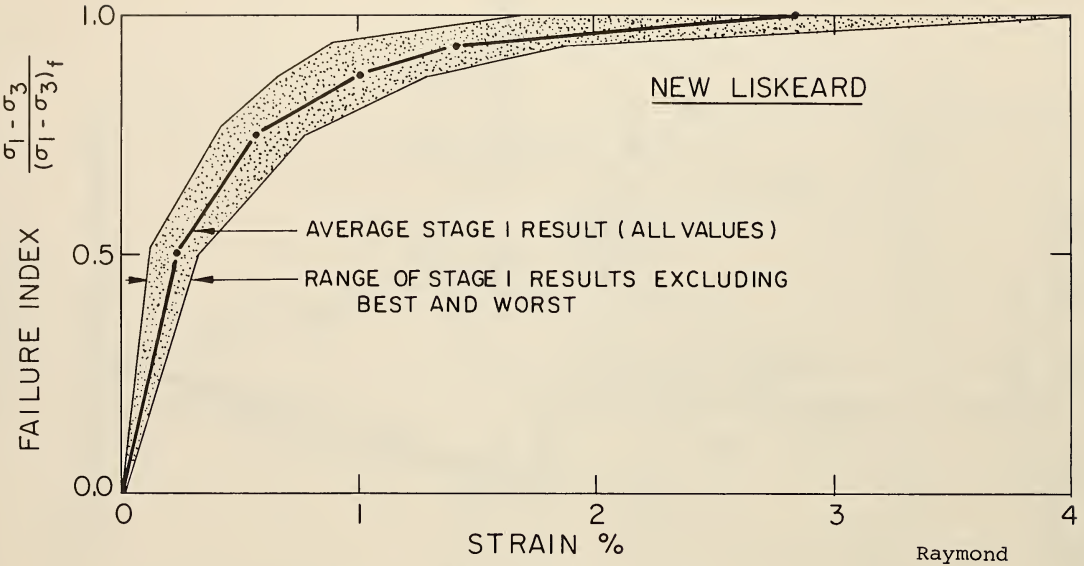

VANE SHEAR STRENGTH kg/cm²
0
.2
.4
.6
.8
1.0
1100 lb/ft²
-10
-20
-30
-40
-50
-60
-70
-80
-90
-100
ELEVATION (ft.)
GIVEN AS STIFF BLUE CLAY
SOFT CLAY
ASSUMED SOIL PROFILE
○ ℄ 1973 V-2
△ 160 FT W ℄ 1973 V-1
□ 160 FT E ℄ 1973 V-3
Raymond
STA 263+00 I-95

FIGURE 4-25.



Raymond

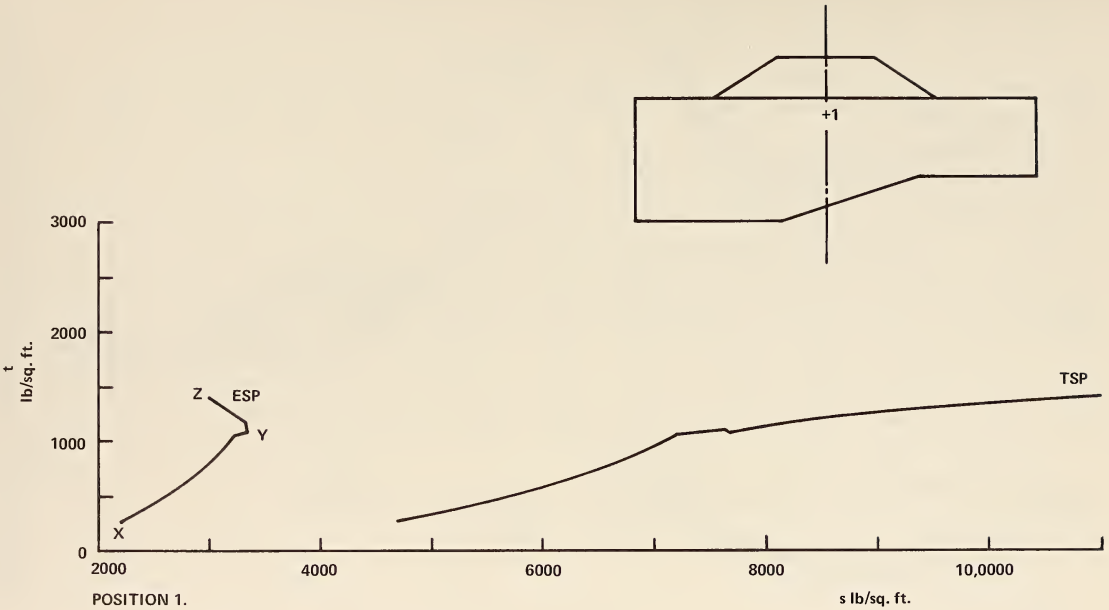
FIGURE 4-26.



STRAIN FITTED TO MIT STAGE I & 2 PERFORMANCE INDICATES

$$\text{BOSTON CLAY STRAIN} \doteq \frac{\text{NEW LISKEARD STRAIN}}{1.5}$$

FIGURE 4-27. NEW LISKEARD



Wroth

FIGURE 4-28. TOTAL AND EFFECTIVE STRESS PATHS FOR ELEMENT 1 UNDER CENTRE LINE OF EMBANKMENT

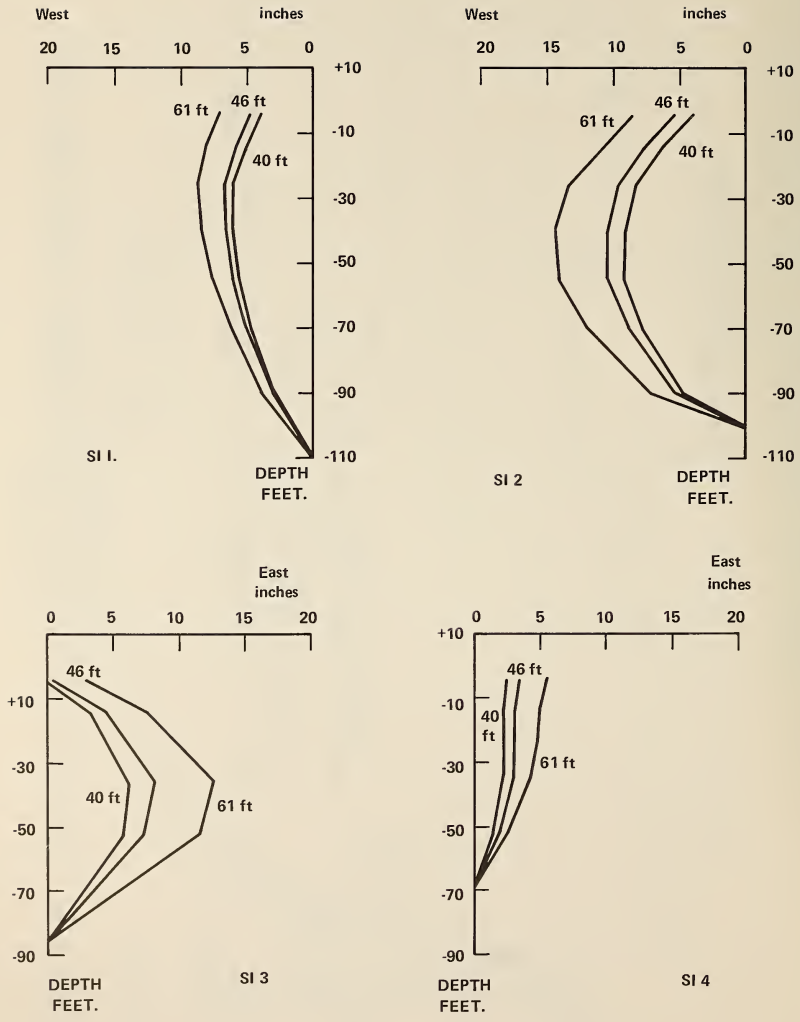


FIGURE 4-29. COMPUTED HORIZONTAL MOVEMENTS OF INCLINOMETERS

Wroth

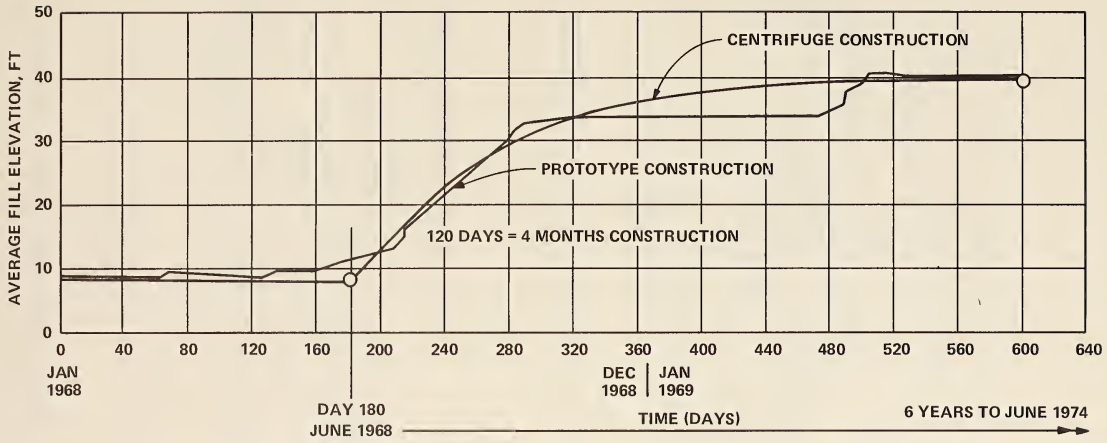


FIGURE 4-30. AVERAGE FILL ELEVATION VS DEPTH I-95 REVERE-SAUGUS, MASS.

Bassett

FIGURE 4-31. VERTICAL DISPLACEMENTS LOAD INCREMENT II
Units - model units $1x10^{-1}$mm ≡ 1.8cm (0.71 inch) prototype

FIGURE 4-32. HORIZONTAL DISPLACEMENTS LOAD INCREMENT II
Units - model units $1x10^{-1}$mm ≡ 1.8cm (0.71 inch) prototype

Bassett

FIGURE 4-33. VOLUMETRIC STRAIN DEVELOPED DURING LOAD INCREMENT. II

FIGURE 4-34. SHEAR STRAIN DEVELOPED DURING LOAD INCREMENT/II

Bassett

V. PRESENTATION OF FIELD MEASUREMENTS

5.1 Failure

Early in the morning of 20 September 1974, a failure of extraordinary proportions occurred. Within minutes, a simultaneous failure to both sides of the embankment caused the crest to drop about 30 feet and the sides to heave as much as 14 feet. An aerial view of the slide area is shown in Figure V-1. Unfortunately, no one witnessed the failure. No surface cracking was noticed the previous day, nor was a clear indication of impending failure obtained from the field instrumentation. The test section failed at an elevation of 56.5 feet, after placing 18.7 feet of additional fill on the existing embankment.

Additions of fill to the test embankment had proceeded much according to plan. The first three construction days were used to steepen the east slope to approximately 40° by end-dumping fill from the embankment crest. Fill was moved with trucks and the embankment crest was kept level with a rubber-tired loader. Figure V-2 shows the progress of filling which proceeded at an average rate of 1.5 feet per day, excluding weekends. The average density of place fill was 122 lb/ft^3 as obtained from field density tests during filling (see Table V-1).

5.2 Field Measurements

5.2.1 Pore Pressures

Figures V-3 and V-4 present some of the pore pressure data obtained during the field test. The computation of excess

pore pressure is based on a water table elevation of +2.5 ft and an artesian pressure in the till of 5 feet of water.

5.2.2 Settlements and Heave

Figure V-5 is a plot of settlement vs. embankment elevation for the four settlement plates located at Station 263. Since the surveys were conducted around the middle of the working day, the embankment elevation selected for the settlement and heave plots was the average elevation during the day of the measurement.

Figure V-6 shows the vertical movement of heave points H-1 and H-2 at Station 263. The offset for H-1 was 130 feet east and for H-2 183 feet east.

5.2.3 Horizontal Movements

Figures V-7a and b show the measured horizontal displacement at inclinometers SI-3 and SI-4. The 8 September 74 plot corresponds to movements measured two days after an additional 6.6 ft of fill had been in place for two days. The arrows in the figures indicate the required predictions.

5.2.4 Summary of Measurements

Table V-2 contains the measurement of each value that the predictors were requested to predict. Also shown are the values predicted by each team. Two values are given for the measured pore pressures at failure--the values obtained the evening preceeding failure and the values obtained one to two hours after failure.

5.3 Failure Description

A surface examination immediately after the failure indicated cracks and movements on the embankment crest and sides from Station 257 to Station 267 +30. Thus failure occurred for a length of 1030 feet. To the south of the test section, the embankment crest was at its original 40 ft elevation minus a settlement of the order of two to three feet. To the north, however, the 40 ft high embankment continued only to Station 266 after which the embankment elevation decreased to +25 ft.

Failure occurred to both sides as shown in Figure V-8. A single frame of an unsuccessful time-lapse film made during the failure indicates failure occurring to both sides simultaneously and movements occurring rapidly. Also shown in Figure V-8 are the depths at which the inclinometers ruptured as indicated by probing after the failure and the zones of maximum horizontal strain up to the day before failure.

At Station 263, the top of the embankment dropped a distance of about 30 feet. A heave of 14 feet was measured on the east side and 10 feet on the west side. The movements to the east side of the embankment were very uniform as shown in Figure V-9. A very large crack (14 ft wide at the top and 8 ft deep) formed on the east side as shown in the photographs of Figures V-10 and V-11 and schematically in Figure V-9.

TABLE V-1. FIELD DENSITY TESTS

TEST NO.	DATE	SAMPLE ELEV.	DRY DENSITY lb/ft^3	WATER CONTENT, %	WET DENSITY lb/ft^3
1	5 Sept 74	44'	118.0	5.79	124.5
2	5 Sept 74	44'	115.5	4.92	121.5
3	5 Sept 74	44'	102.5	3.63	108.5
4	6 Sept 74	43.4'	128.0	7.05	137.0
5	6 Sept 74	43'	127.5	7.5	137.5
6	6 Sept 74	43'	122.5	3.95	127.0
7	13 Sept 74	48.7'	102.0	4.54	109.0
8	13 Sept 74	48.7'	108.0	2.57	111.0

TABLE V-2. MIT FOUNDATION DEFORMATION PREDICTION

Summary of Predictions

ITEM PREDICTED	PREDICTED VALUE										
	DUNCAN	MOH	KIRBY	LADD	MOORE	POULOS	RAYMOND	WROTH	McGUFFEY	BASSET	ACTUAL
Additional height of a fill, T, necessary to cause a stability failure (feet)	12	9	23.5	23	13	27	8	21	13.5	13	18.7
Additional horizontal movement due to adding 6 ft. of fill: @ 6 days (inches)											
at SI-3, Elev. -30	1.0	1.5	1.2	0.7	.75	.17	8.6	1.6	.25	0.6	0.5
at SI-3, Elev. -70	0.5	1.25	0.8	0.35	.75	.016	8.6	0.6	.25	2.0	0.3
at SI-4, Elev. -30	0.5	1.0	0.6	0.45	.75	.47	5.3	0.7	.25	2.5	0
at SI-4, Elev. -70	0	0	0	0	.50	0	0	0	0	2.25	0
Additional settlement of SP-1 due to adding 6 ft. of fill (inches)	2.0	3.0	5.0	0.8	1.25	.76	13.7	2.6	3.0	4.75	0.66
Additional pore pressure due to adding 6 ft. of fill (ft. of water)											
at P-3	19.0	9.5	12	14	8.8	8.8	26	9	7.3	6	9.4
at P-4	21.0	9.2	12	14	7.2	9.5	22	9.6	7.2	8	6.4
at P-6	13.0	11.5	2.5	10	4.8	8.1	11	5.8	4.9	4	5.7
Additional heave due to adding 6 ft. of fill (inches)											
at H-1	0.5	.25	--	0.5	1.0	.73	4.8	0	1.0	2.0	-.12
at H-2	1.0	1.25	--	0.4	0.5	.32	6.2	0	1.0	4.0	-.15
Additional pore pressure at failure (ft. of water)											
at P-3	38.0	12.8	25.5	37	16.6	16.0	29	27	14.8	Lost	★
at P-4	42.0	12.5	25.5	39	16.6	23.1	24	28.3	13.9	15	
at P-6	26.0	17.1	--	25	11.9	14.0	13	7.9	7.6	4	

	10 hrs. before failure	1 hr after failure
★ P-3	21.9	Lost
P-4	21.8	67.5
P-6	10.9	18.8

FIGURE 5-1. AERIAL VIEW OF THE I-95 TEST EMBANKMENT AFTER FAILURE

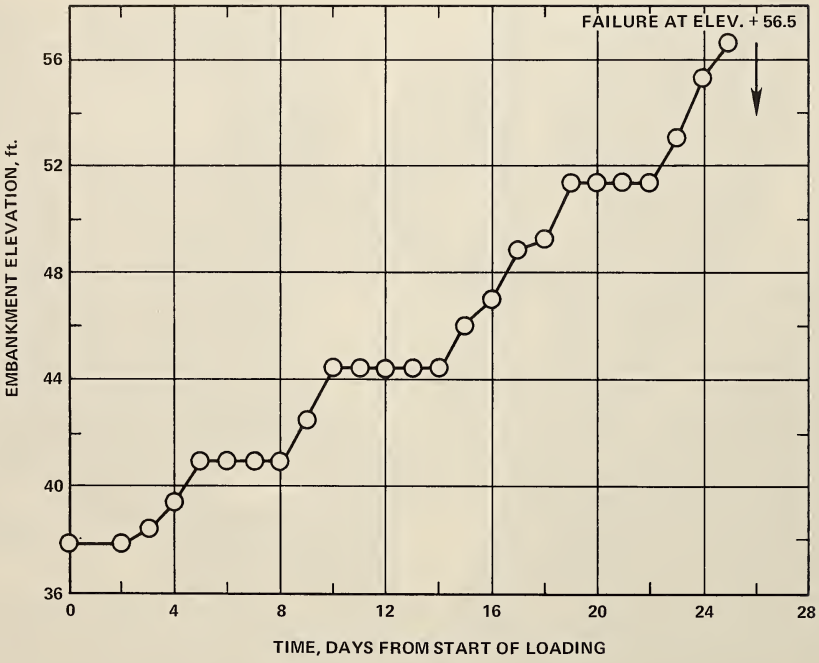


FIGURE 5-2. EMBANKMENT ELEVATION VS TIME STATION 263 TEST SECTION

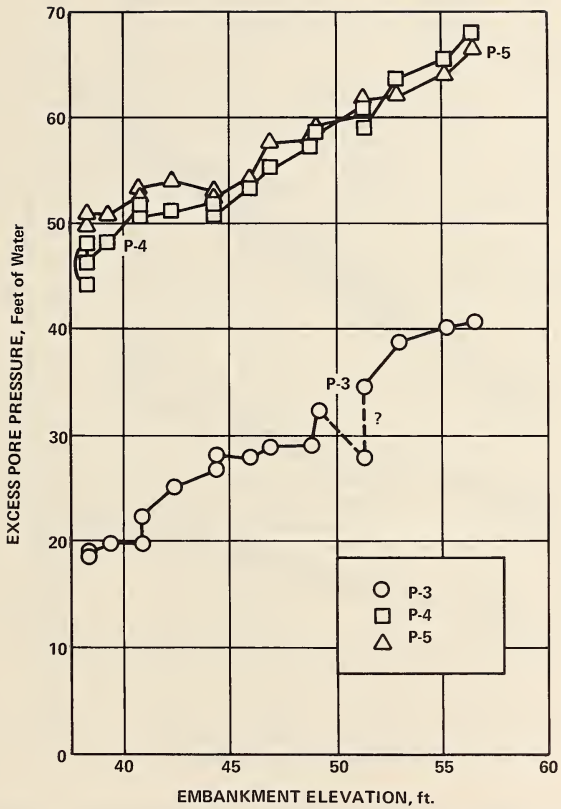


FIGURE 5-3. EXCESS PORE PRESSURE VS EMBANKMENT ELEVATION. STATION 263 TEST SECTION

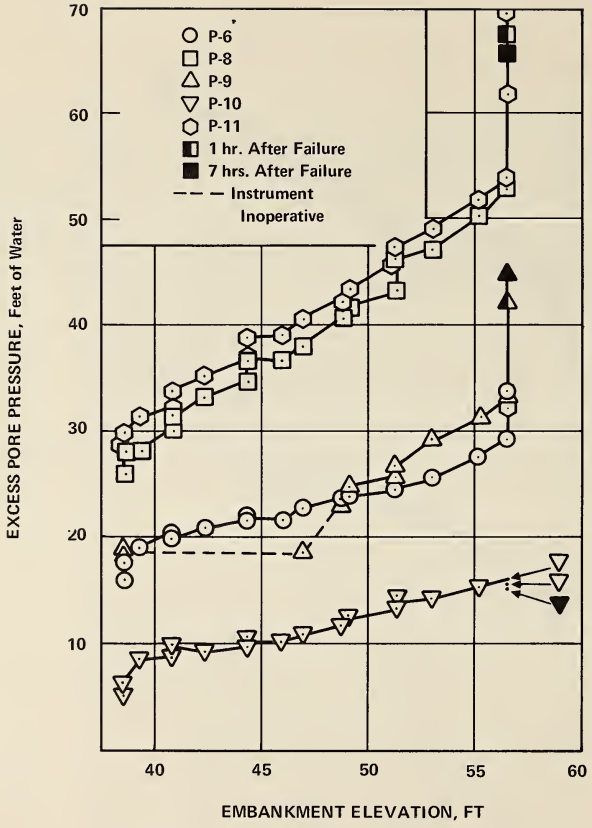


FIGURE 5-4. EXCESS PORE PRESSURE VS EMBANKMENT ELEVATION. STATION 263 TEST SECTION

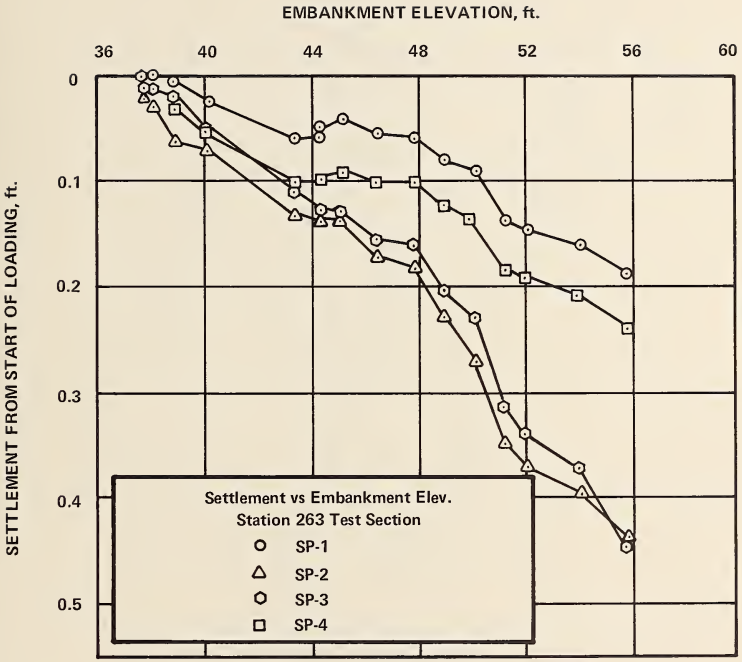


FIGURE 5-5. SETTLEMENT VS EMBANKMENT ELEVATION I-95, STATION 263 TEST SECTION

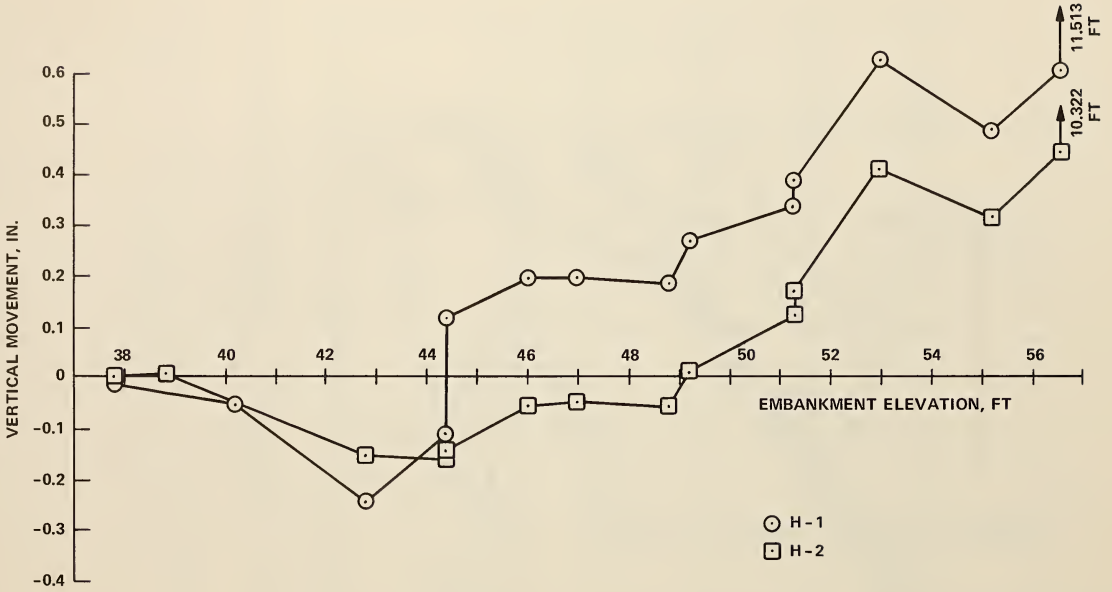


FIGURE 5-6. VERTICAL MOVEMENT VS EMBANKMENT ELEVATION (East Side)

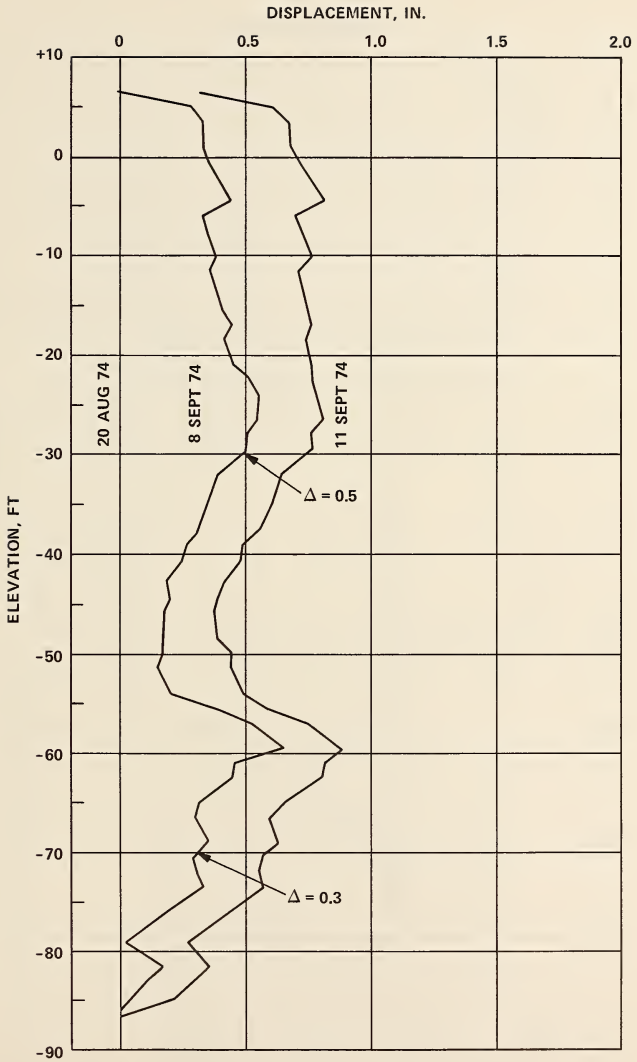


FIGURE 5-7a. ADDITIONAL HORIZONTAL DISPLACEMENT AFTER 6 FEET OF FILL, SI-263-3

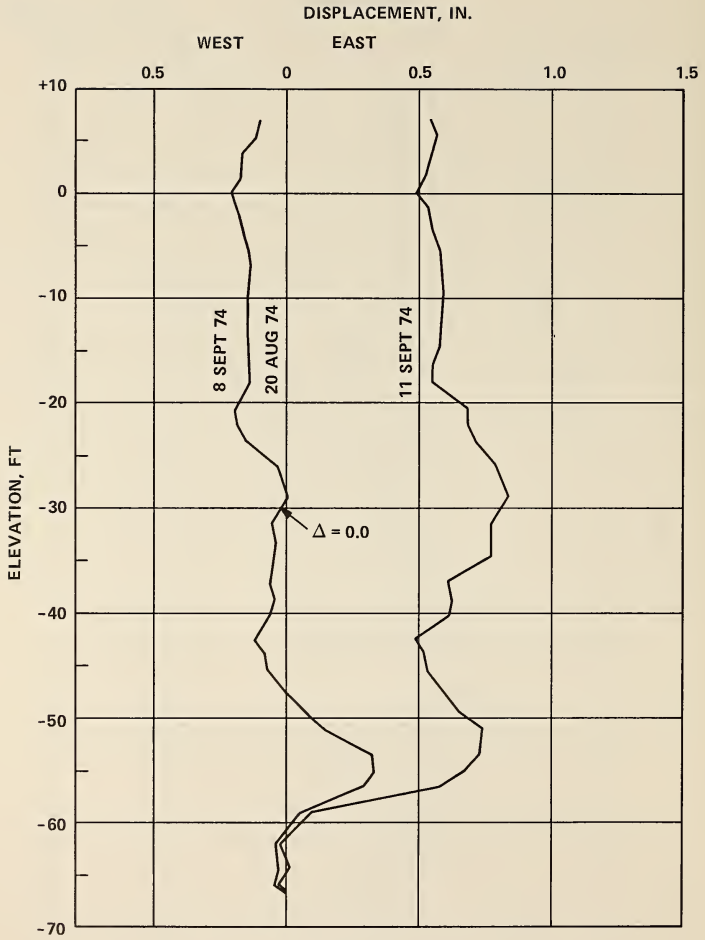


FIGURE 5-7b. ADDITIONAL HORIZONTAL DISPLACEMENT AFTER 6 FEET OF FILL, SI-263-4

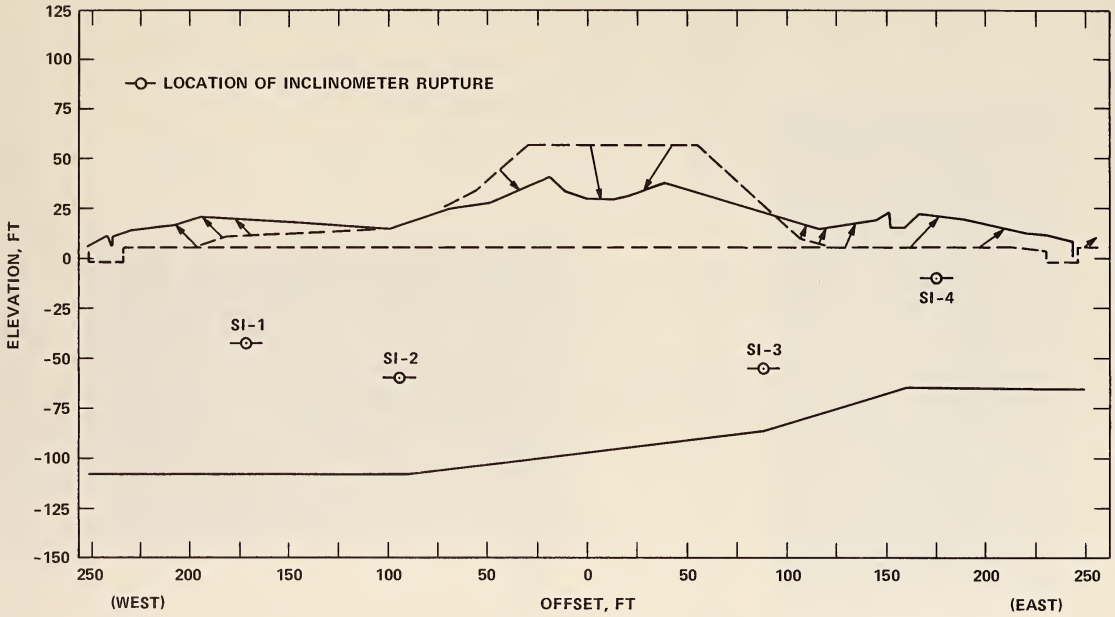


FIGURE 5-8. STATION 263 TEST SECTION BEFORE AND AFTER FAILURE

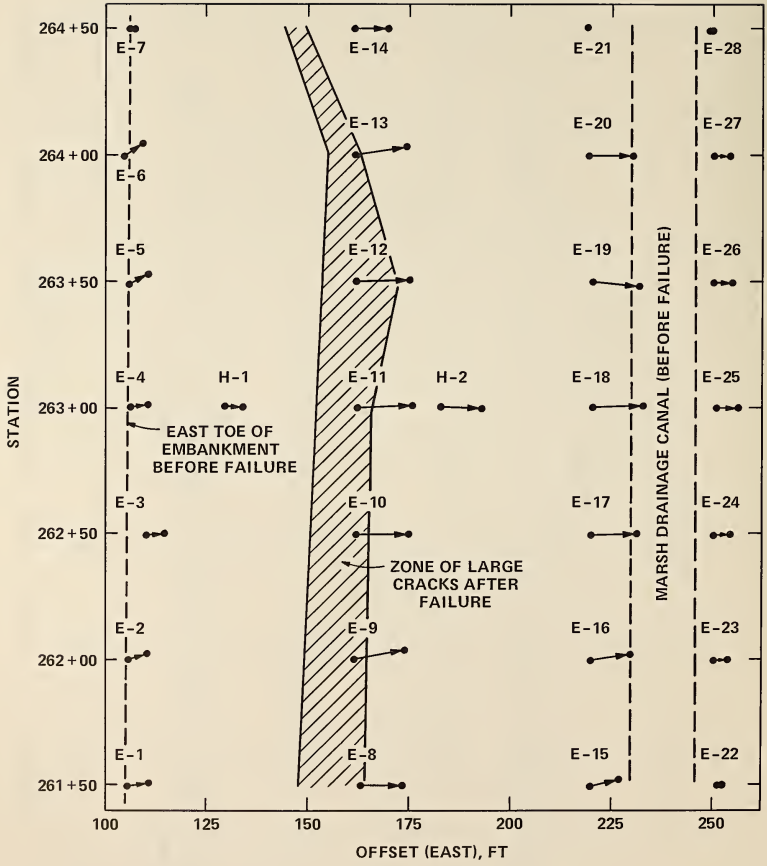


FIGURE 5-9. HORIZONTAL DISPLACEMENT OF SURFACE STAKES

FIGURE 5-10.
STATION 263 TEST SECTION AFTER FAILURE (East Side)

FIGURE 5-11. LARGE CRACK 140'
EAST OF EMBANKMENT CENTERLINE AT STA. 263

VI. DISCUSSION OF RESULTS AND PREDICTIONS

The following comments were made by the predictors immediately after Dr. Marr's presentation of the actual field results. During this period, the predictors were asked to describe their immediate thoughts regarding any possible changes or alterations they would make in their prediction methods based on the actual field results. These presentations were limited to approximately three to five minutes each and the predictors went in order of their original deliveries. The first predictor was Dr. Harry Poulos.

Dr. Poulos would have used the same analyses with one exception: He would not have trusted the finite element stability analysis as much as he had. He had a crude prediction based on a bearing capacity calculation of 22 ft which was significantly better than his 27 ft submitted prediction. He felt there was not adequate data on the undrained modulus especially stress-strain time properties. He would have liked to have seen subsurface earth movement measurements in order to check the validity of the overall results and also the mechanisms. It also would have been helpful to have pore pressure readings in the till. A slightly more detailed instrumentation set-up would have avoided the situation where two people arrive at the same answer, which is the correct answer, by using different assumptions and different methods, and having no way of checking out which one is, in fact, correct.

The next predictor to speak was Bill Moore of Dames and Moore. Mr. Moore was pleased to see some attention given to

the creep phenomenon which has bothered him for quite some time. He thinks that it has more to do with soil behavior than we recognize at this time. He raised the question that if the fill had been stopped at some lower elevation and then left for several months, would it in fact, have failed. Mr. Moore felt the entire exercise was most constructive especially because it emphasized the requirement for good data and the need to understand soil behavior. Mr. Moore would like to see laboratory data that is more reflective of the actual site conditions. He questioned some of the field procedures that were not well-documented. For example: the settlement of SP1 and whether it was affected by the removal of peat and backfilling which apparently was not well-documented.

The next predictor to speak was Professor Charles C. Ladd. Professor Ladd said that he would, in the future, stick to heights of fill and deformations and avoid comments on pore pressure. Professor Ladd still could not understand why the embankment did not fail at a higher elevation.

The next predictor, Doctor R. H. Bassett, supported Mr. Moore's comment regarding the strong possibility of creep failure had the embankment been stopped at a lower elevation and permitted to stand. He would have liked to have had in situ block samples of the material to test in his centrifuge. Boston Blue Clay, with its sensitivity of 3 to 5, has a rather stiff stress-strain response and it builds up rather abruptly. When the clay begins to fail, we find that finite element

programs can not deal effectively with it. Dr. Bassett is not sure in fact, whether his centrifuge can deal effectively with the sensitive clay. He is fairly sure that if he had built the embankment to a height of 15 ft and left it for a period of nine months, it would have failed some time in the period of 0 to 9 months. He also agreed that there is something peculiar regarding creep, especially for sensitive clay, that requires more serious and more extensive investigations.

Dr. Za-Chieh Moh of the Asian Institute of Technology would, if he had to do it over again, stick to a simple slip-circle analysis but put more emphasis on the proper soil parameters and the proper unit weights of materials. He blames MIT, and particularly Allen Marr, for not furnishing adequate data for the problem especially the field densities; the AIT team was lead to believe that it was much lower than 122 lbs/cu ft and strength data was also inadequate. He stresses very much that it is not how sophisticated your analyses are but that you have accurate, reliable strength data. He expresses doubt that on a real practical engineering job, he would have enough data to justify using the Finite Element Method in his analysis. For predicting movements, however, he feels that the Finite Element Method is the only method to use. They had asked for extension test data and for creep data and could have probably made a better estimate of movements had they had this data.

The next speaker was Mr. Verne McGuffey of the New York State Highway Department. Mr. McGuffey felt that with enough study of the data, Professor Ladd's normalized properties concept and the limiting equilibrium analysis, you can come fairly

close to a good estimate of stability for this problem. He was particularly impressed with some of the finite element work that predicted lateral deformations. This work could have been better had they had better data. He also felt that the unit weight of material especially the fill in place perhaps was a little low. In fact, that after compaction, it could be as high as 128 or 129 lbs/cu ft. For the settlement of SP1 at 6 ft of fill, they used an extension of the settlement that had taken place from the prior 6 ft of filling. After studying the data available, they concluded that there was some organic material left in place below the embankment and it would have consolidated during the period from 1969 to 1974. Therefore, under the new loading, it would be in secondary compression and would behave differently from the earlier settlement rate. They estimated on the basis that the peat was still in virgin compression, which was in fact, incorrect and they plan to change that when they return to their offices.

The next speaker was Professor Duncan from the University of California at Berkeley. His comments concerned the fact that both Professor Ladd and he were using Bishop's Modified Method and essentially the same way of evaluating the shear strength. If so, then why did Professor Duncan calculate 12 feet when Professor Ladd calculated 23? This difference must result from differences in the details of the calculations. Professor Duncan used an average value of undrained strength

whereas Professor Ladd gave much more weight to the strength of the material below the central part of the embankment. The use of an average strength had seemed like small detail when he was making the prediction but in fact, on second thought, it appeared to him that it would be more appropriate to give greater weight to the behavior of the clay beneath the center of the embankment. Professor Duncan's second point was that, if you take the extreme heights of fill at failure predicted, the actual variation is only plus or minus 20% from the actual failure elevation. He suggested that this was a more realistic way of looking at the numbers, rather than comparing the additional height of fill. He expressed the opinion that our predictive ability for stability is in fact quite good, and he congratulated MIT on both their testing and their ability to perform analyses. He again emphasized the importance of creep, especially relative to movements. Had he waited a length of time with the six feet of fill in place, the movements would have been considerably larger. We don't know how to scale creep effects from the lab to the field and we don't know how to subtract the creep effects out of measured movements. Creep is an important factor, not only in stability calculations but in analyzing field movements and in interpreting lab test results and projecting them to the field.

The next speak, Mr. Robert C. Kirby, supported Prof. Duncan's observation that the differences in predictions were not large when one considers the total height of fill to cause failure rather than the incremental height of fill to cause

failure. He believes that the most important result of the failure is a back-calculated undrained shear strength for Boston Blue Clay. Mr. Kirby's analyses (and those of other predictors) showed one could predict a wide range of failure heights depending on what shear strength profile is used.

There is a difference in philosophy between analysis and design. We were not asked to design an embankment on soft clay but rather to predict the failure of an embankment on clay. For design of this embankment, one would probably determine shear strength from unconfined compression tests and estimate the design elevation of the embankment to be equivalent to a loading of about six times the undrained shear strength using a factor of safety of 1.1. If the embankment had to be higher than this, one could make field vane tests in order to better determine shear strength. However, the design elevation of the embankment would be determined using a higher factor of safety. If one were designing for stability based on consolidated, undrained, triaxial, compression tests, an even higher factor of safety would be used when arriving at the design fill elevation.

When hindcasting measured performance using the Finite Element Method, one can "predict" almost anything depending on the parameters which are input to the model. There are at least eight or ten parameters which can be adjusted and the answers are controlled by the mix of parameters one chooses. Mr. Kirby feels that some of the judgments made relative to selection of parameters for finite element solutions were not

reasonable because they were not supported by the available data.

The next predictor that spoke was Professor Gerald Raymond from Canada. He said that the vane strengths were lower on the west side than on the east side and so he did feel it could fail both ways. Professor Raymond would have placed a lot more emphasis on vane tests. He feels that lateral movements are important and should be measured and understood and you have to go to a finite element analysis to deal with these horizontal movements. He is still modifying his model which is similar to the Cambridge model. Professor Raymond feels that if you are going to put numbers into a computer program, you may as well use a complex computer program.

The next predictor to speak was Doctor Peter Wroth from England who indicated that some of his colleagues thought he was mad to get involved in such a symposium. He complimented MIT on the conduct of the entire effort especially the actual causing of the failure within the specified time limit. Dr. Wroth said that from the engineering standpoint, there are two processes involved. One is prediction and design and the other is analysis and understanding: they are distinctly different processes. Regarding the first of these, they had used elastic-plastic models of soil behavior and were particularly interested in the predictions of excess pore pressures. He was very satisfied with his finite element predictions. He felt it would be interesting to ask the predictors to resubmit predictions of movements and pore pressures for the actual height of the embankment at failure rather than the (different) heights

that each had predicted would cause failure. Regarding the second process of analysis and understanding, he considered that we should now see if any change of behavior of the soil from elastic to plastic had occurred prior to failure. This could be done by a careful evaluation of some of the field measurements.

Next, Prof. Lambe asked the predictors to comment on whether they were surprised at the extent of the failure, in other words, its propagation both north and south from the loaded area.

Dr. Wroth was surprised! He said that most methods of analysis were two-dimensional and did not consider the effect of the length of propagation. If asked whether the sliding mass would propagate beyond the loaded area, he would have said yes, but certainly nothing like the extent to which it had happened.

Prof. Duncan felt that the extent of the failure was one of the big mysteries of the entire symposium. He is not sure why it failed in such a fashion.

Dr. Bassett was not surprised at the extent of the failure and again, postulated that the very high pore pressures generated within the failure zone could probably be transmitted quite rapidly in a horizontal direction and cause additional failures.

Dr. Wroth felt that some work should be done to try to estimate the amount of energy that was dissipated as the failure propagated away from the loaded zone. Is this eneryg transmitted by means of the high excess pore pressures, or is it transmitted by means of the intermediate effective stress, $\bar{\sigma}_2$?

Professor Raymond was concerned that the embankment would fail in both directions and felt vindicated by the results.

Mr. McGuffey described the recent failure of about 1,000 ft of interstate embankment that propagated an additional 2000 ft. One of the safety factors in the failure zone was about .95 where is was overstresses and calculated 1.4 in the other zones prior to failure. He suggested that the high-pore pressure is generated at the failure zone and can, in fact, travel quickly, rapidly, and at great distances in a lateral direction. Prof. Raymond feels that the sensitivity of the clay had something to do with the propagation of the failure zone beyond the loaded zone. Dr. Bassett felt that perhaps the clay is laminated and then can transmit these pore pressures laterally very quickly. Professor Duncan said that he sees very little use for predicting pore pressures for this type of problem--that movements can be predicted and stability analyzed without predicting pore pressures.

Professor Ladd said that an effective stress analysis at failure is absolutely worthless.

Dr. Wroth emphasized that his finite element analysis is an effective stress analysis. Since it is effective stresses that control the behavior of the soil, it is those stresses that we should be studying. Dr Wroth said that as long as you are studying pre-failure conditions, it was proper and appropriate to use effective stresses, but that conditions at and post failure might be better predicted in terms of total stresses

because of the difficulties of predicting post-failure pore pressures.

Prof. Moh also agrees that the lateral extent of a failure is related to pore pressure transmission, Mr. Kirby said that, based on his experience, limiting equilibrium analyses using measured pore pressures and effective stress parameters do not give satisfactory predictions. Mr. Moore also agrees with Prof. Duncan in that pore pressures are rather useless to understand the failure conditions.

Dr. Bassett also felt that his increase in pore pressure did propagate and did in fact, contribute to the failure on either side of the loaded section. Professor Raymond agrees with that statement although he says creep is also very important in this regard. Mr. McGuffey said that he does believe in creep and that they have had progressive failures in New York. Failures have taken place approximately a month after the end of filling. Calculated factors of safety were about 1.3. Mr. McGuffey says that they run special tests in New York on strains on the order of 20% and can modify normal stress-strain relationships when they find areas in an embankment analysis that become overstressed. These areas' strength is then reduced before performing the stability analysis and thus a more realistic value of safety factor is obtained by using these reduced strengths in areas that have been overstressed.

At this point, the discussion focused on the differences between and importance of creep and progressive failure. Dr. Wroth said that his understanding of creep is deformation that occurs under a constant effective stress whereas for progressive failure, the effective stresses are changing.

Professor Raymond said that undrained creep would in fact generate pore pressures; therefore, the effective stress would not be constant. Mr. Moore said that the issue is a very important one as time dependent creep may occur at very low stresses. So, Mr. Moore says we are now again worrying about stress levels that could cause slow movements that would then continue on to failure which is sort of the same thing that we worried about 30 years ago under the name of yield point stress.

Professor Lambe then asked the panel how they would control construction if they wanted to build this embankment as quickly as possible and as high as possible while keeping the factor of safety very slightly over 1. Professor Ladd would use inclinometer measurements to control construction because he feels that lateral deformations are the best indicator of impending failure.

Professor Duncan says that if you ran a total stress analysis, then you must use movements to control construction and he feels that movements are a very difficult way to do this. The reliability of movement predictions is far less than the reliability of predictions of stability. Mr. Moore said that he feels movement measurements are very important and they do this frequently. Especially, he looks at the relationship of volume change due to vertical movement and horizontal movement. They use a combination of vertical movements and horizontal movements to note whether they are getting consolidation and increase in strength in addition to other phenomena. Mr. Moore also said it is important to evaluate the time rate

of movement. Mr. Kirby feels that controlling construction with field measurements is very difficult; there is no one field measurement which conclusively indicates imminent failure. Measurements of settlement and horizontal movement are useful indicators. In addition, he would recommend pore pressure measurements. A plot of total head vs. fill elevation should be made for each piezometer. A change in slope of the data from one straight line segment to another indicates the soil at that location has yielded. We should not be concerned when the soil beneath the center of the embankment yields locally but we are concerned when the yielded zone approaches the toe of the embankment. Consequently, piezometers should be installed near the toe of the embankment. Dr. Moh would definitely use piezometers to monitor pore pressures and an effective stress analysis and therefore to keep calculating stability as the embankment grew. Dr. Moh says that if you measure lateral movements, by the time you got significant ones, it would probably be too late.

Professor Lambe asked the panel whether any of them would have or could have used decision theory or probability in coming up with their prediction. Dr. Wroth was the first to respond. Before we could adopt such theories, we must have some parameters that we have confidence in, or a certain degree of confidence. Dr. Wroth was grateful to MIT for exposing him to the need to make use of probability and decision theory and he would make an effort to consider it in detail.

Mr. McGuffey felt that you could not apply theories of probability and risk analysis in a case where life is in danger that in fact, there you must have 100% reliability.

Mr. Moore felt that in a typical project, you rarely have enough data to apply statistical probability theory. Mr. Moore also felt that in engineering, there is no such thing as 100% safety.

Professor Ladd had a comment regarding probability in that there is some bias introduced because of the nature of the test data. One can run several kinds of strength tests and get very different answers. We then have to be able to resolve the fact that there is uncertainty in the measured strengths. So, relating this further to probability of failure becomes very difficult.

Dr. Bassett had a comment that stems from a job he did in the United Kingdom where he installed 500 soil anchors and tested them all to 1.5 times working stress and failed some 39 of these anchors. His standard deviation band was much wider than he had predicted. It very strongly suggested to him that on a job where there are a large number of repetitive operations of a similar nature, that we cannot very easily decide what is or what is not reasonable to expect for a standard deviation. He also added that use for probability should probably be restricted to jobs that are much like mechanical engineering where we have this repetitive action that we can apply the theory of probability to.

Prof. Raymond was not very enthusiastic about statistics and probability again claiming that the primary reason is the lack of sufficient data to call it a representative sample.

After the discussion on decision theory, Prof. M. M. Baligh of MIT discussed the three-dimensional aspects of the failure. In the initial formulation of the problem itself, Dr. Marr had considered the length of the loaded section in order to achieve a plane strain situation. The extent of the failure beyond the limits of the loaded area indeed surprised Prof. Baligh. Prof. Baligh reviewed the literature and found very few three-dimensional slope stability analyses that responded to the needs of this particular problem. Prof. Baligh considered several geometric shapes, simple cylindrical slip surface with straight ends and then with elipsoid and conical shaped ends trying to find the variations with safety factor versus some of his geometric parameters. Prof. Baligh concluded that the end effects increase the factor of safety obtained by means of a conventional 2-D solution by about 19% for the I-95 test fill. In addition, he said that even though the end effects on the factor of safety can be predicted with reasonable accuracy, the length of failure is very sensitive to the parameters defining the problem and is thus difficult to predict.

After Prof. Baligh's presentation, there were questions for the panel from the floor. These questions are briefly summarized below.

Prof. Selig asked that if they were building this embankment from scratch, starting from ground zero and building up,

would they have used the vane values and had confidence in them? Dr. Poulos said he would not use the vane shear values without trying to correlate them with some type of lab data. Professor Duncan said that he was very surprised that the UU tests gave strengths that were only 40% of the strength determined using CU tests for Boston Blue Clay. Professor Duncan said on a design job, he might use the vane shear strength. Mr. Kirby responded that if you had no prior experience in the area and only vane shear strength values, you would have to be satisfied using a rather high safety factor, perhaps as high as 1.6. Prof. Raymond said that in Canada they put a lot of faith in vane strength values, design many embankments which in fact stand up if, of course, the vane test is run properly. He added that many engineers prefer a field test over a lab test. Dr. Wroth thought that sampling disturbance was always a problem and that in particular, the unconfined compression test was a complete waste of time and effort.

Mr. McGuffey basically doesn't use the U and UU tests. He feels that you can get as much information and more reliable and consistent information with the pocket penetrometer. Mr. Kirby was asked a question about safety factor chosen as a function of the type of strength data. He pointed out that most of our experinece is based on U and UU tests and that the factor of safety adopted for design is based on case histories where U and UU tests were used. More recently, we are attempting to use some of the newer tests that give different (and usually higher) shear strengths. We don't have a long history of experience with these test results and must reconsider the safety

factor that we use. In his view, a higher safety factor is required for design when using the more sophisticated laboratory tests. Prof. Ladd reminded those present that throughout the world, we don't always get a lower strength value with a UU test. The Boston Blue Clay samples we used were taken from very large depths and therefore we had much lower strengths due to excessive disturbance.

The question was asked "if they had to design another embankment adjacent to this one, what safety factor and method would they use?" Dr. Bassett responded that he would use a Bishop's analysis with vane shear strengths and no correction factor. Mr. McGuffey would have used a safety factor of 1.25 with the same strength parameters. Dr. Wroth would use vane shear strengths but he would ask the client what the consequences of failure would be before choosing a safety factor. Prof. Raymond would have used the Bishop analysis, vane shear strengths and a safety factor of 1.15. Professor Duncan said that he would use the SHANSEP stress parameters with a creep correction and a safety factor of 1.15. Dr. Poulos would have used a Bishop analysis, SHANSEP strength parameters with a correction for anisotropy and a safety factor of 1.3. Mr. Kirby would run field vanes along the alignment of the new embankment and if they were no lower than the previous one, he would not do any analysis and would tell the client he could build to elevation 40 with no trouble.

A question was asked from the audience: "Is there any way to predict the increase in pore pressure outside the failure zone?" Mr. Kirby said that he doesn't feel that pore pressures transmitted laterally caused the additional failure. He feels that the ends of the embankment were just dragged down by shear stresses and then the failure propogated farther and farther away from the original failure area. Prof. Raymond reminded people that the actual loaded area was probably too short to be considered

plane strain. There was a very strong possibility that the initial failure would have been experienced at a lower height than the 18.5 feet which caused the failure at our particular case. Robert Tierney of the MDPW asked the panel "If we decide that we are going to build I-95 and we now need this road, what do we do with our completely remolded foundation?" Prof. Raymond said that it will get large differential settlements in this area because it is remolded and it has a heavier load on top of it. Mr. McGuffey says that he agreed that it would get large differential settlements and therefore he would place several berms and put another surchage load on the area to try to even up some of these zones and build a highway. Dr. Bassett felt that he wouldn't dare put a surcharge load on this area and that he would build a road and just plan to resurface it every six months or so until the large settlement stopped. Dr. Marr felt that even after the failure, we are still above the elevation of the future road and that if it were not built up for six months to a year, we would actually be taking material off at that time and have had some further consolidation and settlement that would help us.

A question was asked by an engineer from TAMS, Ernest Jonas, whether the strength of the fill was considered in the analyses, since there was reasonable height of fill and shearing took place in the fill. Prof. Raymond said that he did not consider the strength in his analyses. Joe Olsen of MIT asked the question "If we had this embankment, and the problem was, instead of trying to fail it to put on an additional height of fill that was to last for a minimum of 25 years, if we considered creep

phenomena, what is the height of fill that the panel would agree to place on it if it had to last 25 years?" Mr McGuffey said 13 feet Dr. Basset said 13 feet also. Prof. Raymond would go between 8 and 13 feet. Dr. Wroth said that he could pick 14 ft out of the air, but that the phenomenon of creep raised very complex problems that should be looked at in detail. Prof. Duncan said that he feels that the creep problem does go quickly so he probably wouldn't add more than about 5 feet of fill to the embankment. Prof. Ladd would backfigure the strength for the 18.7 feet and then use a safety factor of 1.3 to reduce the height. Dr. Moh would have put on about 8 or 9 feet. Mr. Kirby said he would not know how to answer that question. Mr. Kirby is not convinced that we have a creep problem with Boston Blue Clay. He can easily predict the amount of consolidation settlements that are required to fit the problem; therefore, in his opinion, creep is not a major problem here.

Professor Duncan and Mr. Kirby got into a discussion about how much of the time-dependent settlement was due to consolidation and how much was due to creep.

Mr. McGuffey agrees with Professor Duncan in that we cannot explain all of the settlements by using consolidation settlements. The discussion was cut off at this point and Prof. Lambe delivered a short summary prior to the closing of the symposium.

VII. CONCLUDING REMARKS

Professor Lambe felt that the symposium turned out very well. He said that we always have good presentations when we get able people to focus on a single problem.

We started out with the premise that: predictions constitute an important, a very essential part of Civil Engineering. Making the prediction is very necessary, but insufficient. The selection of parameters is very important and is the major limitation in making good predictions. That the accuracy of the prediction is directly related to the sophistication of the analysis is a totally erroneous statement. We should leave this symposium understanding that fact.

There are three classes of predictions: the crude prediction, the engineering prediction and the state-of-the knowledge. We do not necessarily get a more accurate prediction if we use a state-of-the-knowledge prediction. There are two reasons for making the state-of-the-knowledge prediction: first, it gives an insight into the fundamentals of the problem and second, it gives a good basis for a future evaluation. It is very difficult to use the crude analysis and then make an evaluation of the prediction.

Prof. Lambe sees the finite element technique as being a very powerful tool and playing two important roles: 1) to generate parametric studies that can look at a problem in several different ways and 2) to provide a very powerful

analysis tool for the state-of-the-knowledge type analysis. He feels that the average engineer will not have the time or money to use a finite element analysis on the first cut.

Prof. Lambe was impressed with the centrifuge test. However, he felt its main use will be identifying mechanisms and not solving practical problems by making numerical predictions.

He was impressed with the extrapolation technique which is the simplest way to make a prediction. We can extrapolate to get stresses, movements, pore pressures, but we cannot extrapolate very well to get factor of safety. Experience is very important but probably one's own experience has the most significance. Sometimes in engineering it is rather difficult to read of the experiences of others and then relate to them accurately. For experience to be useful, it must be evaluated experience. If you have done the same thing over and over again for twenty years, you have twenty years' experience, but you are probably not much better off than you were at the end of the first year.

The symposium has raised a number of important questions and Prof. Lambe felt that as we think about these questions hopefully we can come up with better explanations than those offered at the sumposium. Why the embankment failed over such a large area was not answered properly.

Prof. Lambe concluded the symposium with thanks to the predictors for the preparation and presentation of their predictions.

VIII. BIOGRAPHICAL NOTES FOR PREDICTORS

Richard H. Bassett
Department of Civil Engineering
University of London King's College

Dr. Bassett graduated from King's College, London in 1958. After graduation, he joined Binnie and Partners working on the design of a number of major hydroelectric and water supply projects in Great Britain and the Commonwealth. In 1961, he went to Cambridge University to work for a doctorate degree under the late Professor Roscoe on the Cambridge stress-strain theories. From 1964 to 1969, Dr. Bassett worked as a consultant and was responsible for the foundation work and design of several major dam and power station projects. Since 1969, Dr. Bassett has held research and teaching positions, first at Cambridge and then, from 1972, at King's College, London continuing to work on embankment model tests, ground anchors and retaining walls.

Edward W. Brand
Associate Professor
Chairman, Division of Geotechnical Engineering
Asian Institute of Technology

Dr. Brand received his B. Sc. and Ph. D. from Leeds University. He is the Editor of the journal, Geotechnical Engineering. Dr. Brand has served on the faculty of AIT for several years. His research interests range from soil properties to foundation engineering.

Edward H. Davis
School of Engineering
The University of Sydney
Sydney, Australia

Dr. Davis was educated at University College School and University College in London. After a period at the Road Research

Laboratory in the UK and a lectureship at University College, he joined the staff on the University of Sydney in 1952, becoming a Professor of Civil Engineering in 1968. He is a Fellow of the Institution of Engineers, Australia and is one of the founders of the Australian Geomechanics Society. He is active in the affairs of the International Society for Soil Mechanics and Foundations having been a Vice-President of that Society. He has spent periods of study leave at MIT, the Building Research Station in the UK and Imperial College, London. The two main areas of his research are: the development of a unified theory of the behavior of foundations and the use of the theories of elasticity and plasticity for soil and rock stability problems.

James Michael Duncan
Department of Civil Engineering
Unviersity of California, Berkeley
Berkeley, California

Professor Duncan received his BSCE in 1959 and his MSCE in 1962 from the Georgia Institute of Technology. In 1965, he received his Ph. D. from the University of California at Berkeley. He was a Project Engineer with Delta Engineering Co. from 1959-60 and a Civil Engineer in the Soils Division of the US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi in 1962. He joined the staff at the University of California, Berkeley in 1964. Professor Duncan is currently Chairman of the Committee on Embankment Dams and Slopes of the American Society of Civil Engineers. In 1972, he was the recipient of the ASCE Collingwood Prize, and in 1973 he was awarded the Walter Huber Research Prize by ASCE. His main research interests involve finite element

modeling of stress-strain behavior of soil, and analysis of slope stability problems. He is a registered Civil Engineer in California.

Raymond L. Gemme
Senior Soils Engineer
Bureau of Soil Mechanics
New York State Department of Transportation
Albany, New York

Mr. Gemme received his BSCE from Northeastern University in 1963 and his MSCE from Rensselaer Polytechnic Institute in 1969. Mr. Gemme has worked as a soils engineer in the New York State Department of Transportation, Soil Mechanics Bureau for the past 11 years, seven of which he has been a supervisor of design squads in the Roadway Design Section. He is a licensed Professional Engineer and a Licensed Land Surveyor in New York State.

J.M.O. Hughes
University of Auckland
New Zealand

Dr. Hughes received his Civil Engineering degree in 1964 and Master's degree in Soil Mechanics in 1965 from the University of Auckland. From 1965-67, he was employed in structural design by Beca, Carter, Hollings & Ferner, Auckland, New Zealand and from 1967-69 he was employed in foundation design by R. C. Thurber & Associates, Victoria, British Columbia, Canada. He received his Ph. D. from Cambridge University in 1973 and remained for a year at Cambridge after receiving his degree continuing his research in in situ measurement of soil properties. He is now a Senior Lecturer at the University of Auckland.

R. C. Kirby
Woodward-Moorhouse & Associates, Inc.
Clifton, New Jersey

Mr. Kirby received his BSCE in 1969 from Northeastern University. In 1972, he received his MSCE and Civil Engineer's degree from Massachusetts Institute of Technology. He joined Woodward-Moorhouse in 1972, where his assignments have included design of building foundations, slope stability analyses, liquefaction analyses, and the design and implementation of field measurement systems. While at MIT, Mr. Kirby concentrated on evaluating methods of predicting the performance of a well-instrumented embankment on soft soil and developing a design methodology for embankments on soft soil. Mr. Kirby is a registered Professional Engineer in New York.

Charles Cushing Ladd III
Department of Civil Engineering
Massachusetts Institute of Technology

Professor Ladd received his AB from Bowdoin College and a SB degree in Building Engineering and Construction from MIT in 1955. He received his SMCE in 1957 and his Sc. D. in 1961 in Soil Engineering from MIT. He has been on the MIT teaching staff becoming full Professor in 1970. His research interests have focused on the engineering properties of soils and soft ground construction. He has been consultant to T. William Lambe and Associates, Haley and Aldrich, Inc. and Dames & Moore on major projects involving buildings, dams, highways, flood control facilities and an offshore nuclear power plant. He received the ASCE Research Prize in 1969 and the Croes Medal in 1973 and is active in the Boston Society of Civil Engineers, American Society

Charles Cushing Ladd (Con't)

of Civil Engineers and the Transportation Research Board. He is a registered Professional Engineer in Massachusetts.

Verne C. McGuffey
Associate Soils Engineer
Bureau of Soil Mechanics
New York State Department of Transportation
Albany, New York

Mr. McGuffey received his BSCE in 1956 and his MSCE in 1958 from Rensselaer Polytechnic Institute. Mr. McGuffey has been a practicing soil engineer in the New York State Department of Transportation, Soil Mechanics Bureau for 16 years and supervisor of Roadway Foundation Section and Laboratory for approximately 10 years. He is a registered Professional Engineer in New York.

Za-Chieh Moh
Professor of Geotechnical Engineering
Vice-President and Provost
Asian Institute of Technology
Bangkok, Thailand

Dr. Moh received his BSCE from National Taiwan University, his MS from Iowa State University and his Sc. D. from Massachusetts Institute of Technology. After serving on the faculty at Yale University, Dr. Moh went to the Asian Institute of Technology. He is now Professor of Geotechnical Engineering and Provost of the Asian Institute of Technology. He is the Vice-President for Asia for the International Society for Soil Mechanics and Foundation Engineering

William W. Moore
Dames & Moore
San Francisco, California

Mr. Moore received his BSCE in 1933 and his MSCE in 1934 from the California Institute of Technology. He worked for a period with R. V. Labarree and Professor F. J. Converse. He also worked for the Corps of Engineers and for the US Coast and Geodetic Survey on an investigation of earthquake effects. Since the founding of the firm, he has been a senior partner of Dames & Moore and has been in responsible charge of projects covering all aspects of the earth sciences. These projects include foundation engineering services for more than 100 major refinery installations, 50 land development projects involving the placing of hydraulic fill, extensive studies of both vibration and blast effects, the effects of earthquake forces, and the development of offshore drilling platforms in California. Mr. Moore is a licensed Professional Engineer in the states of California, Hawaii, Nevada, Oregon, Washington, Wyoming and New York.

Harry G. Poulos
School of Engineering
The University of Sydney
Sydney, Australia

Dr. Poulos received his BE and Ph. D. degrees in civil engineering from the University of Sydney, Australia. In 1965, after working for a firm of consulting engineers for a year, he joined the Civil Engineering Department at Sydney University, where he is now a reader. He spent 1969 at MIT as a Fullbright Scholar. His research concerns the settlement of foundations and behavior of piles. Dr. Poulos was co-recipient of the 1972 Warren Memorial Prize of the Institute of Engineers, Australia, and the 1973 J. James R. Croes Medal, awarded by the ASCE.

Gerald Patrick Raymond
Department of Civil Engineering
Queen's University
Kingston, Ontario, Canada

Dr. Raymond received his B. Sc. in Civil Engineering from the University of London in 1956, his MSE in Soil Mechanics from Princeton University in 1957 and his Ph. D. (1965) and D. Sc. (1973) in Soil Mechanics from the University of London. From 1959 to 1961, Dr. Raymond was a Lecturer at the University of Sydney in Australia. Since 1962, he has been a member of the faculty of Queen's University, Canada, currently as Professor of Civil Engineering. He has broad research interests ranging from development of new theories of consolidation to methods of design for construction on soft clays and peat.

Archie M. Richardson
Associate Professor of Geotechnical Engineering
Asian Institute of Technology

Dr. Richardson received his BS and MS from the University of Pittsburgh and his Sc. D. from Massachusetts Institute of Technology. He was associated with A. C. Ackenheil & Assoc., Inc. He has served on the faculty of the University of Pittsburgh and universities in Afghanistan and India.

C. P. Wroth
University Engineering Department
Cambridge, England

Dr. Wroth received his BA in 1952, MA in 1955, and Ph. D. in 1958 from the University of Cambridge. From 1961 to the present he has been a Lecturer in Engineering at the University of Cambridge. He spent 1967-68 on sabbatical leave as a Visiting Professor at Cornell University. Since 1970, he has been Head

C. P. Wroth (Con't)

of Soil Mechanics Sub-division of the Engineering Department, University of Cambridge. His research interests have focused on development of stress-strain laws for soils, new methods of in situ measurement of soil properties and foundation engineering.

REFEREES

David S. Gedney
Regional Engineer
Region 15 - Arlington, Virginia
Rederal Highway Administration

Mr. Gedney received his BSCE from Tufts University in 1959 and his MSCE in Soil Mechanics from Purdue University in 1960. He is a registered Professional Engineer in Maryland. From 1960 to 1964, Mr. Gedney was a soil engineer with the firm of Haley & Aldrich, Inc. in Cambridge, Massachusetts; from 1964 to 1968 he was the Regional Soils and Pavement Design Engineer, Bureau of Public Roads for Region 2; and from 1968 to 1973 he was Chief of the Soil and Rock Mechanics Branch, Construction and Maintenance Division of the Office of Highway Operations, Federal Highway Administration. He was appointed to his present position in August 1973.

Robert T. Tierney
Chief Engineer
Massachusetts Department of Public Works

Mr. Tierney received his BSCE from the University of Notre Dame in 1949. He attended Harvard University Graduate School studying Soil Mechanics in 1965. He spent 1949-50 as an Engineer

Robert T. Tierney (Con't)

in training with the Ohio Department of Highways. Since 1950, Mr. Tierney has been with the Massachusetts Department of Public Works, becoming Chief Engineer in 1972. He is a registered Professional Engineer and Land Surveyor in Massachusetts.

IX. ROSTER

Massachusetts Institute of Technology — Department of Civil Engineering

FOUNDATION DEFORMATION PREDICTION SYMPOSIUM

Nov. 13-15, 1974

ROSTER

Tim G. Abrams
National Soil Services, Inc.
5814 Heffernan
Houston, TX 77017

Alberto Amadeo-Lopategui
President
Metropolitan Soils
G.P.O. Box 2291
San Juan, PR 00936

Juraj J. Bajgar
Metcalf & Eddy, Inc.
Statler Bldg.
Boston, MA 02116

Alexander K. Barsvary
Section Head
Ministry of Transportation
Geotechnical Office
1201 Wilson Ave.
Downsview, Ontario
CANADA

R. Biegstraaten
Rijkswaterstaat
Deltadienst
Van Alkemadelaan 400
Den Haag, HOLLAND

Jan Boehmer
Rijkswaterstaat
Deltadienst
Van Alkemadelaan 400
Den Haag, HOLLAND

W. Bronkhorst
Rijkswaterstaat
Deltadienst
Van Alkemadelaan 400
Den Haag, HOLLAND

John H. Bryant
Sr. Soils Engineer
Stone & Webster
110 Milk Street
Boston, MA

Richard Buser
JFK Building
FHWA
Boston, MA

Chien-Tan Chang
Highway Engineer
Office of Development FHWA
HDV-22 Implementation Div.
Washington, DC 20590

Ron Chassie
Dept. of Transportation
FHWA
400 7th Street SW
Washington, DC 20590

Wayne W.H. Chen
Sargent & Lundy Engrs.
55 E. Monroe Street
Chicago, IL 60603

Norman J. Coons
President
N.J. Coons & Assoc.
387 Passaic Ave.
Fairfield, NJ 07006

David H. Connell
Woodward-Lundgren & Assoc.
P. O. Box 24075
Oakland, CA 94623

Claude Crepeau
Research and Materials Div.
MDPW
99 Worcester Street
Wellesley Hills, MA 02181

Robert Cunny
U. S. Army Corps of Engrs.
Waterways Experiment Station
P. O. Box 631
Vicksburg, MS 39180

Umakant Dash
Penn. Dept. of Transportation
P. O. Box 2926
Harrisburg, PA 17105

John Driscoll
Research and Materials Div.
MDPW
99 Worcester Street
Wellesley Hills, MA 02181

William J. Eden
Div. of Bldg. Research
National Research Council
Ottawa, CANADA K1A OR6

Victor Elias
The Reinforced Earth Co.
1414 22nd Street, NW
Washington, DC 20418

John J. Erglis
Tippetts-Abbett-McCarthy
Stratton
345 Park Ave.
New York, NY 10022

Raymond R. Garneau
Terratech Ltd.
275 Benjamin-Hudon
Montreal, CANADA H4N1J1

John W. Guinnee
Transportation Research
Board
2101 Constitution Ave., NW
Washington, DC 20418

David P. Hammer
Chief, Soils Section
U. S. Army Corps of Engrs.
Savannah District
P. O. Box 889
Savannah, GA 31402

Jeng-Rong Hsu
Ohio State University
Civil Engrg. Dept.
2070 Neil Ave.
Columbus, OH 43210

Richard Ierardi
Research and Materials Div.
MDPW
99 Worcester Street
Wellesley Hills, MA 02181

Richard Isa
Box 61970
Caracas 106
VENEZUELA

Stanley J. Johnson
Special Asst.
Soils & Pavement Lab
Waterways Experiment Station
Vicksburg, MS 39180

Ernest Jonas
Tippetts-Abbett-McCarthy-
Stratton
345 Park Ave.
New York, NY 10022

John S. Jones, Jr.
Law Engrg. Testing Co.
7913 Westpark Dr.
P.O. Drawer QQ
McLean, VA 22101

Demetrius C. Koutsoftas
Project Engineer
Dames & Moore
6 Commerce Drive
Cranford, NJ 07016

Francis L. Krieger
A.M. Kinney, Inc.
2900 Vernon Place
Cincinnatti, OH 45219

Suzanne M. Lacasse
Ecole Polytechnique
Genie Civil
P.O. Box 6079, Succ. A
Suite B-334
Montreal, Quebec H3C 3A7
CANADA

Robert M. Leary
New Hampshire Dept. of
Public Works & Highways
Materials and Research Div.
33 Stickney Ave.
Concord, NH 02201

Rolf Leininger
John P. Stopen, C.E.
103 E. Water St.
Syracuse, NY 13202

Richard P. Long
Dept. of Civil Engineering
University of Connecticut
U-37
Storrs, CT 06268

John E. Lyons
Exxon Research & Engrg. Co.
P. O. Box 101
Florham Park, NJ 07932

Andrew F. McKown
Geotechnical Engineers, Inc.
1017 Main Street
Winchester, MA 01890

Robert E. McPhail
Golder Gass Assoc.
1033 Mass. Ave.
Cambridge, MA 02138

Louis P. Mishu
Tennessee State University
Civil Engineering Dept.
Nashville, TN

Wayne W. Moky
Harza Engrg. Co.
150 S. Wacker Dr.
Chicago, IL 60606

Jerome C. Neyer
Halpert, Neyer & Assoc.
29226 Orchard Lake Rd.
Farmington, MI 48024

Charles Owen, Jr.
The Architects Collaborative
46 Brattle Street
Cambridge, MA 02138

Hiram Pearlman
Division Bridge Engineer
FHWA
612 JFK Federal Bldg.
Boston, MA 02203

George R. Powledge
Chief, Foundations &
Materials Branch
Engrg. Div. U.S. Army Engr.
District
P.O. Box 1229
Galveston, TX 77550

Terje Preber
Dames & Moore
1550 NW Highway
Park Ridge, IL 60068

Joseph J. Rixner
Haley & Aldrich, Inc.
238 Main Street
Cambridge, MA 02142

Philip Robinson
JFK Building
FHWA
Boston, MA

Wendel T. Ruff
Mississippi State
Highway Dept.
Testing Division
P.O. Box 1850
Jackson, MS 39206

Surenda K. Saxena
Dames & Moore
6 Commerce Dr.
Cranford, NJ 07016

Harcharian Singh
Dames & Moore
Bethesada, MD

Nassef N. Soliman
Dames & Moore
6 Commerce Dr.
Cranford, NJ 07016

Antonio P. Soriano
Davis Hall
University of California
Berkeley, CA 94720

Michel F. Soulie
Ecole Polytechnique
Genie Civil, Succ. A
P.O. Box 6079
Suite B-339
Montreal, Quebec H3C 3A7
CANADA

William Sparacin
School of Civil and
Environmental Engrg.
Cornell University
Ithaca, NY 14850

James K. Stangenberg
C.E. Maguire, Inc.
60 First Ave.
Waltham, MA 02154

Harvey H. Stoller
Fay, Spofford & Thorndike, Inc.
11 Beacon Street
Boston, MA 02108

J. Allen Tice
Law Engineering Testing Co.
P.O. Box 18288
Raleigh, NC 27609

Fernando H. Tinoco
Universidad Simon Bolivar
Apartado 5334 - Sartenejas
Caracas
VENEZUELA

Harvey E. Wahls
North Carolina State Univ.
Civil Engineering Dept.
Box 5993
Raleigh, NC 27607

Walter C. Waidelich
FHWA
4 Normanskill Blvd.
Delmar, NY 12054

John L. Walkinshaw
Dept. of Transportation
FHWA
400 7th Street, SW
Washington, DC 20590

M. James Warren
Manager, Sand Drain Div.
Vibroflotation Foundation Co.
930 Ft. Duquesne Blvd.
Pittsburgh, PA 15222

Charles E. Williams
Woodward-Krazynski & Assoc.
7330 Westview Drive
Houston, TX 77055

Stanley Woolman
Research Manager
FHWA
4 Normanskill Blvd.
Delmar, NY 12054

Tien H. Wu
Ohio State University
Department of Civil Engrg.
2070 Neil Ave.
Columbus, OH 43210

Tai-Sung Yoo
Dames & Moore
6 Commerce Dr.
Cranford, NJ 07016

Lloyd W. Young, Jr.
Bechtel, Inc.
P.O. Box 607
Gaithersburg, MD 20760

PREDICTORS:

SYMPOSIUM PARTICIPANTS

Richard Bassett
University of London King's College
Strand London WC2R 2LS
ENGLAND

Roy Bell
Dames & Moore
500 Sansome Street
San Francisco, CA 94111

Edward W. Brand
Chairman
Div. of Geotechnical Engrg.
Asian Institute of Technology
Bangkok, THAILAND

Edward H. Davis
School of Engineering
The University of Sydney
Sydney, NSW 2006 AUSTRALIA

James Michael Duncan
Department of Civil Engineering
University of California
Berkeley, CA 94720

Raymond L. Gemme
Senior Soils Engineer
Bureau of Soil Mechanics
New York State Dept. of
Transportation --The Campus
Albany, NY 12226

Michael J. Hess
Dames & Moore
500 Sansome Street
San Francisco, CA 94111

J.M.O. Hughes
University of Aukland
NEW ZEALAND

Robert C. Kirby
Woodward-Moorhouse & Assoc.
1373 Broad Street
Clifton, NJ 07012

Charles Cushing Ladd III
Department of Civil Engrg.
M.I.T.
Cambridge, MA 02139

Carl Matuszek
Bureau of Soil Mechanics
New York State Dept. of
Transportation --The Campus
Albany, NY 12226

Verne McGuffey
Associate Soils Engineer
Bureau of Soil Mechanics
New York State Dept. of
Transportation --The Campus
Albany, NY 12226

Za-Chieh Moh
Professor of Geotechnical Engr.
Vice President and Provost
Asian Institute of Technology
Bangkok, THAILAND

William W. Moore
Dames & Moore
500 Sansome Street
San Francisco, CA 94111

Harry G. Poulos
School of Engineering
The University of Sydney
Sydney, NSW 2006 AUSTRALIA

Gerald Patrick Raymond
Department of Civil Engrg.
Queen's University
Kingston, Ontario, CANADA

Archie M. Richardson
Associate Prof. of Geotechnical
Engineering
Asian Institute of Technology
Bangkok, THAILAND

C.P. Wroth
University Engineering Dept.
Trumpington St. CB2 1PZ
Cambridge, ENGLAND

REFEREES:

David S. Gedney
Regional Engineer
Region 15
Federal Highway Administration
Arlington, VA 22201

Robert T. Tierney
Chief Engineer
Mass. Dept. of Public Works
100 Nashua Street
Boston, MA 02114

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